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Goethals on Panama Slides

Report of George W. Goethals, Governor of the Panama Canal, to the Secretary of War; with Selected Illustrations

SYNOPSIS—Present great slides on east and west banks at Culebra involve about ten million cubic yards, of which a million yards per month is being dredged out. Dredging is the only feasible method of attack. Prior slides at other points, including the Cucaracha slide, are discussed. Report written under date Oct. 26, 1915, at Balboa Heights, Canal Zone.

The following report is submitted on the slides which have closed the Panama Canal to navigation. On account of the importance of the subject, I deem it advisable to make a comprehensive report, even at the expense of repetition of the matter contained in previous documents on the subject.

The slides which developed during the construction period were of three distinct types. The first were due to the moving of material from the sides of the prism or cutting, when these were left by the excavation at a steeper slope than that at which the material would remain at rest.

In those of the second type porous material mixed with clay, small rock and spalls, overlay smooth surfaces of rock or other impermeable substances, harder than the overlying mass, which sloped toward the axis of the canal. In excavating the prism through these sections, the removal of the lateral support, combined with the effects of the water permeating through to the underlying inclined strata, resulted in the sliding of the material into the excavated area, the rains assisting and accelerating the movement.

FIG. 1. VIEW NORTH BETWEEN CONTRACTORS HILL (LEFT) AND GOLD HILL (RIGHT) SHOWING CULEBRA REGION

Small slides on both banks caused by admission of water to canal. The great slides came later

The slides of the third class were designated "breaks," and were of two kinds, best described by the localities—those in the vicinity of Culebra and those at or near La Pita Point. In the former the first manifestation was a crack in the surface of the ground, generally parallel to the axis of the canal, with the ends running to the edge of the prism. This cracking was followed by a depression on the side of the crack away from the cut and a movement of the mass between the crack and the canal into the excavated area. The sinking movement on the top was accompanied by an upheaving of the bottom of the canal prism.

The cause of these movements was attributed to the poor quality of the underlying rock, which was intersected by vertical seams or seams sloping toward the canal. As the material was excavated not only was the lateral support removed, but the load distribution was changed to an extent that caused the rupture of the underlying weaker strata. After the break the movement into the cut was like an ordinary slide of the first and second classes, though the cause was entirely different.

The breaks in the vicinity of La Pita Point consisted of the moving or pushing out and a subsequent settling down of sections of rock masses which formed the banks. Three such breaks occurred and were due undoubtedly to water pressure from the Obispo or East diversion channel. This source of trouble has been relieved by turning the waters of the channel into the cut.

The first class of slides gave relatively but little trouble, even during the construction period, and ceased when the material assumed its natural slope. In anticipation of other possible movements of this kind, slopes which seemed too steep have been flattened.

Considerable difficulty was experienced with slides of the second class, and the most notable example was the one at Cucaracha, just south of Gold Hill (Fig. 2). The only method of handling them was to allow the material to enter the prism and remove it from below, for the adjacent ground did not lend itself to handling the material in any other way, and attempts to hold the banks by piling, rock covering or drainage failed. The slide of this class at Cucaracha consisted of clay mixed with rock and rock spalls moving into the cut, the movement resembling the action of a glacier.

History of the Cucaracha Slide

The sliding began during the operations of the French company, and continued to cause trouble until the early part of 1912, when it became quiescent. The material in moving left exposed a mass of rock, which the geologist assured us was of ample strength to hold back the clay and other material remaining on the hillside above it. The indications were that the future trouble in this locality would be limited in extent and caused by small masses of clay washed down by rains or from the usual erosive action to be anticipated in this climate.

FIG. 2. THE GREAT CUCARACHA SLIDE OF 1913, CAUSED BY FAILURE OF ROCK KNOLL
Nearly 4,700,000 yd. was dug out; 1,000,000 yd. more sluiced off the hillside

These expectations and predictions were not realized, however, for on Jan. 20, 1913, following a few days after a rock movement from the south face of Gold Hill, the rock support broke at some part of its structure near or below the bottom level of the canal and a mass of clay and rock moved into the cut, completely filling the prism across its entire width and up to a bench that existed along the west face at the 69-ft. level (see Fig. 2).

The length of the canal blocked by this earth movement was approximately 1,600 ft., and the quantity of material to be removed was estimated at 2,000,000 cu.yd. The slide was attacked by steam shovels located on the 69-ft. level on the west side and on the bottom, or the 40-ft., level. As the excavation proceeded the material broke away from the higher elevations on the face of the hillside and moved down into the prism.

An examination of the top of the hill indicated that there were large masses of clayey material remaining that would eventually move into the cut. The topography of the country lent itself admirably to sluicing operations, by which a large part of the material near the top could be washed into a valley to the east. A sluicing plant was available, and steps were taken to draw off to the east side of the ridge as much of the clay as could be reached by the nozzles of the sluicing plant and thus relieve the situation in the prism itself. This was successfully accomplished.

The balance of the cut, some 8½ mi., was nearing completion; steam shovels were concentrated at Cucaracha slide and as many were worked as available space permitted. In February eight of them removed 168,677 cu.yd.; six shovels working in March and April removed 203,030 and 204,308 cu.yd. respectively; but these were dry-season months. The rainy season reduced the output of the six shovels in May to 148,000 cu.yd. and in June to 124,844 cu.yd. The expectation of reaching the bottom at this time had not been realized. Difficulty was experienced in maintaining tracks, not only at the slide but on the dumps, due to the character of the material that was handled. The cost of removing 921,000 and 808,000 cu.yd. during the last two months respectively from the entire length of the cut, including Cucaracha, was 62 and 66c. per cu.yd. respectively. With Cucaracha alone remaining, the difficulties and cost would be increased.

There was a dredging fleet available capable of handling larger quantities of material than the shovels. The dredges could work 24 hr. a day, while the shovels could not work to exceed 12 hr.; the excavation could proceed without delays due to upset of steam shovels and disarrangement of tracks; the material could be dumped in the Gatun Lake; so that all indications pointed to dredging as the best means, both as to time and money. This course was taken.

The steam shovels operating at Cucaracha slide from Jan. 20, 1913, to Sept. 11, 1913, removed a total of 1,139,708 cu.yd., or an average of 142,463.4 cu.yd. per month. The dredging operations began on Oct. 23, 1913, and up to Aug. 3, 1914, on which date the first boat went through the canal, the dredges removed 2,767,080 cu.yd., or an average of 286,239.78 cu.yd. per month. The cost of the steam-shovel work at Cucaracha was not separated from the remainder of the work in the cut, but the total of 8,225,989 cu.yd. removed during the period under consideration was done at an average cost of 65c. per cu.yd., while the cost of the dredging averaged 44c. per cu.yd. Active operations at Cucaracha slide ceased on Oct. 14, 1914, on which date the total amount removed by steam shovels and dredges since Jan. 20, 1913, was 4,679,238 cu.yd. In addition, upward of

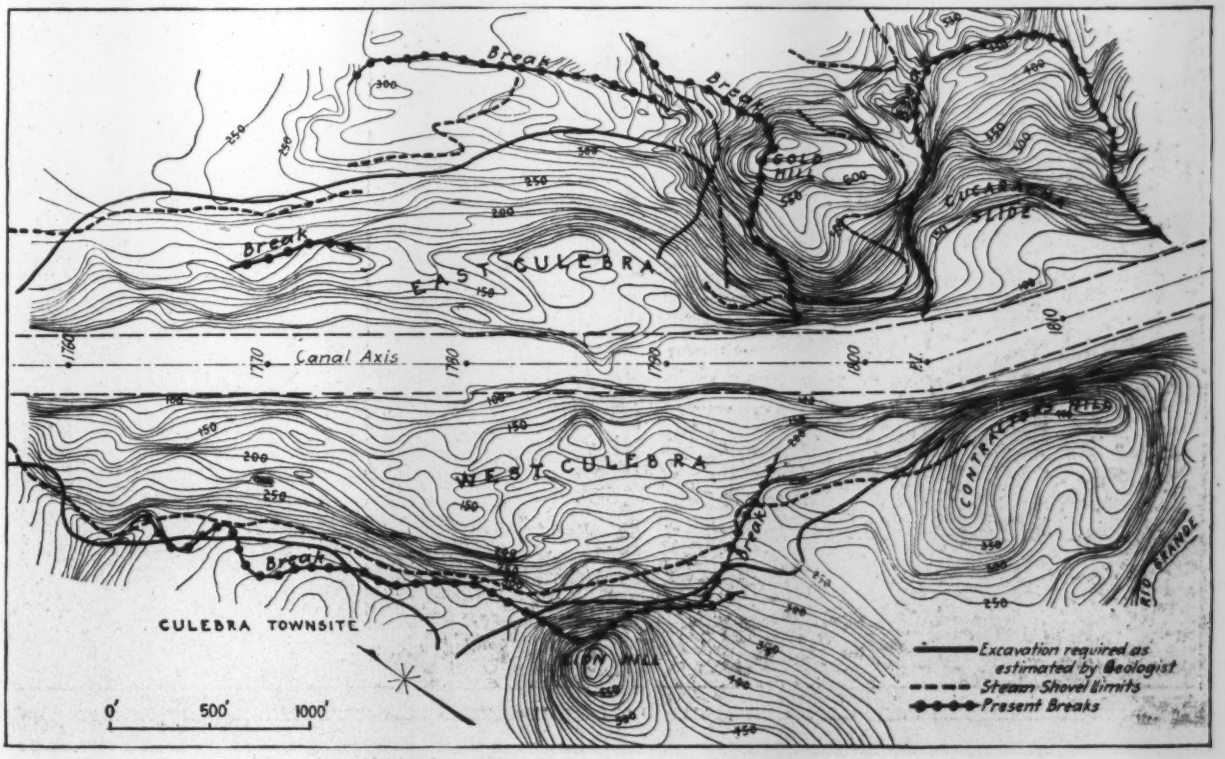


FIG. 3. CONTOUR MAP OF CULEBRA SLIDE OF 1915, AND LOCATION OF BREAKS

FIG. 4. THE CULEBRA SLIDE IN AN EARLY STAGE; SLIDE FROM EAST SIDE ONLY, OCTOBER, 1914

1,000,000 cu.yd. which would have gone into the cut was removed by sluicing, at a cost of 20c. per cu.yd.

The first break in the vicinity of Culebra occurred near the old Panama R.R. station in 1907, and affected but a few hundred feet. Subsequently, as the depth of the excavation increased, the breaks became more extensive, both in length and quantities of material involved; furthermore, they occurred in both the east and west banks, until the upward movement along the bottom of the excavated area affected a length of 2,000 ft. north of Gold Hill. As the cause was the unequal distribution of the pressure exerted by the adjacent banks, it followed that by reducing the height of these banks, the movements would cease or be reduced. Opposition developed to this method on the ground that more material would be removed than was necessary; that by allowing the material that broke off to move into the cut, the minimum amount would be handled, and the material left to itself would assume its natural slope.

Culebra Slides; Their Recent History

Two breaks occurred in rather rapid succession at Culebra in the latter part of 1910. As usual, each completely overturned the tracks at the bottom of the excavation, interrupted drainage and seriously delayed operations in the prism. The latter one of the two breaks

FIG. 6. WAVE OF SLIDING MATERIAL MOVING INTO CANAL PRISM FROM WEST BANK

FIG. 5. FULLY DEVELOPED CULEBRA SLIDE, AFTER AUGUST, 1915, WHEN BOTH BANKS SLID

determined the change in plan, and in January, 1911, the reduction of the height of the adjacent banks on either side of Gaillard Cut in the vicinity of Culebra, was directed and begun. This work continued until December, 1913, at which time 6,533,924 cu.yd. had been removed from the east bank and 8,797,990 cu.yd. from the west bank, resulting in slopes varying from 1 on 1.5 to 1 on 6.5 for the east side and 1 on 2.46 to 1 on 4.35 for the west side, in benches (Fig. 10).

The result of this method of procedure was that when the operations in the cut in the vicinity of Culebra were completed, prior to the admission of water, the breaks in the banks and the upward movement of the bottom had ceased entirely.

Early in 1911 a geologist was employed in connection with the earth movements and proposed maximum and minimum lines for lightening the banks, together with suggested slopes. In the map (Fig. 3) are shown with heavy black lines the maximum probable limits of the earth movement on the two sides, as predicted by him. The map also shows by broken black lines the areas over which the banks were lightened by excavation, and when the shovels were removed the benches within the areas marked by broken lines were all intact, the shovels having

FIG. 7. BROKEN-UP CONDITION OF THE SLIDE MASSES; POOR SURFACE FOR STEAM-SHOVEL WORK

dug so as to remove all evidences of cracks. So far as could be foreseen satisfactory slopes had been secured.

In the early part of 1914 small cracks had developed on some of the upper benches on the west side and similar effects were noted on the east side. During the dry season the surface of the ground in this country cracks as the result of drying out and in some instances these cracks are of considerable depth and extent. Whether those found on the benches were the ordinary dry-season cracks or the result of structural weakness was not known, though they resembled the former. To guard against emergencies, however, the shovels were returned and worked on the east bank, reducing the slope still more, until Apr. 1, 1914; and on the west bank the shovels continued until June 14, 1914.

Suspicious Cracks at Zion Hill and on East Side

A crack had existed for some years at the foot of Zion Hill, southwest of Culebra; the hill was pronounced as geologically secure against any movement, and when the material had been cut away from this bank, the slope of the excavation extended to the crack or the limit marked as the maximum limit by the geologist.

On the east side, toward the end of the dry season of 1913 a crack was found north of Gold Hill, parallel to the cut, about 1,300 ft. back from the face. It was not connected with the slope of the prism at either end. There were no cracks or breaks between it and the cut proper. There was apparently a slight settlement on the side of the crack nearest the cut, but none with the marked indications of a probable break. Steps, however, were taken to lighten the banks between the crack and the cut by sluicing away the high ground and by steam-shovel operations. There were no indications of movement, and apparently the work done toward lightening the banks had been effective. It appeared, so far as could be determined, that equilibrium had been secured.

Great Slide Begins in 1914

Subsequent to turning in the water there were small slides on the east bank, north of Gold Hill, and also on the west side, but they were small, the material was easily removed and they were typical of Class 1 slides.

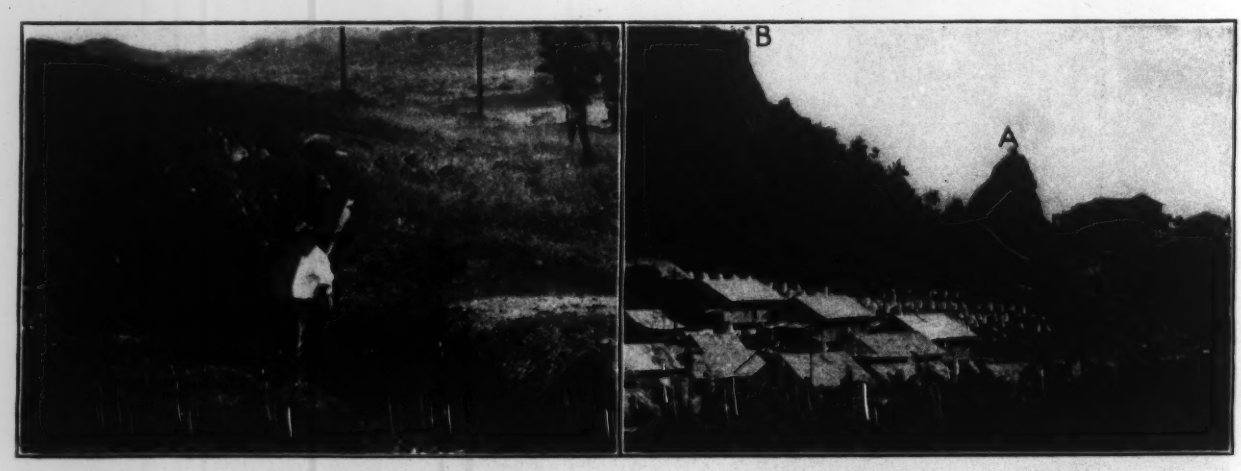
No change occurred until Oct. 14, 1914, when, without warning, a section of the east bank settled vertically 20 ft. This section measured 2,000 ft. along the prism face and extended back about 1,000 ft. from the axis of the canal, generally along an irregular curved line. The bank was from 300 to 350 ft. above sea level, and the extension of the ground eastward was relatively flat country. In the settlement, the upper portion that broke away remained practically parallel to its original position, and the benches which formed the upper part of the slide had not changed their relative positions, though they were badly broken up, while the lower strata were squeezed out across the canal. Subsequently the broken mass moved into the cut (Fig. 4), and until August, 1915, the dredges were able to keep up with the movement, when the movement on the west bank occurred.

A crack was found on the slope of Zion Hill in June, 1914, but observation made upon it showed no movement until subsequent to the break on the east side, when a gradual but general breaking up of the west bank began (Fig. 8). Subsequently the cracks on Zion Hill extended to El. 480, the limit of the present break. The movement into the cut from the west bank occurred early in August, 1915, when a section of Zion Hill broke away and settled down (Fig. 9).

Some Features of the Slide Mass

The break is generally along a curve. The movements from the two sides are toward the central portion of the inclosed area, and at this central portion is the main obstruction to the channel. In coming into the cut the mass on the east side is breaking up into waves, which move down to the prism in succession. One now occupies part of the channel (Fig. 6) and another is forming up the slope of the hill. On the west bank this wave effect is also evident. Here the bank was pushed forward horizontally, the benches in some portions still maintaining their relative position with respect to each other, but the ground in their rear has settled, forming sinkholes on the side away from the prism.

When the break of Oct. 14, 1914, occurred it was evident that the large mass of material that had broken away must be handled by the dredges. Turning any part of the material away from the prism by sluicing operations was not possible on account of the natural lay of the ground, and in fact all material that could be handled by that method had been removed when the



FIGS. 8 AND 9. SLIDE EFFECTS AT ZION HILL, ON WEST BANK AT CULEBRA, IN 1914 AND 1915

Fig. 8—Enlargement of crack on slope of hill in 1914, after break on east side. Fig. 9—Piece "A" of Zion Hill slipped off "B" in August, 1915

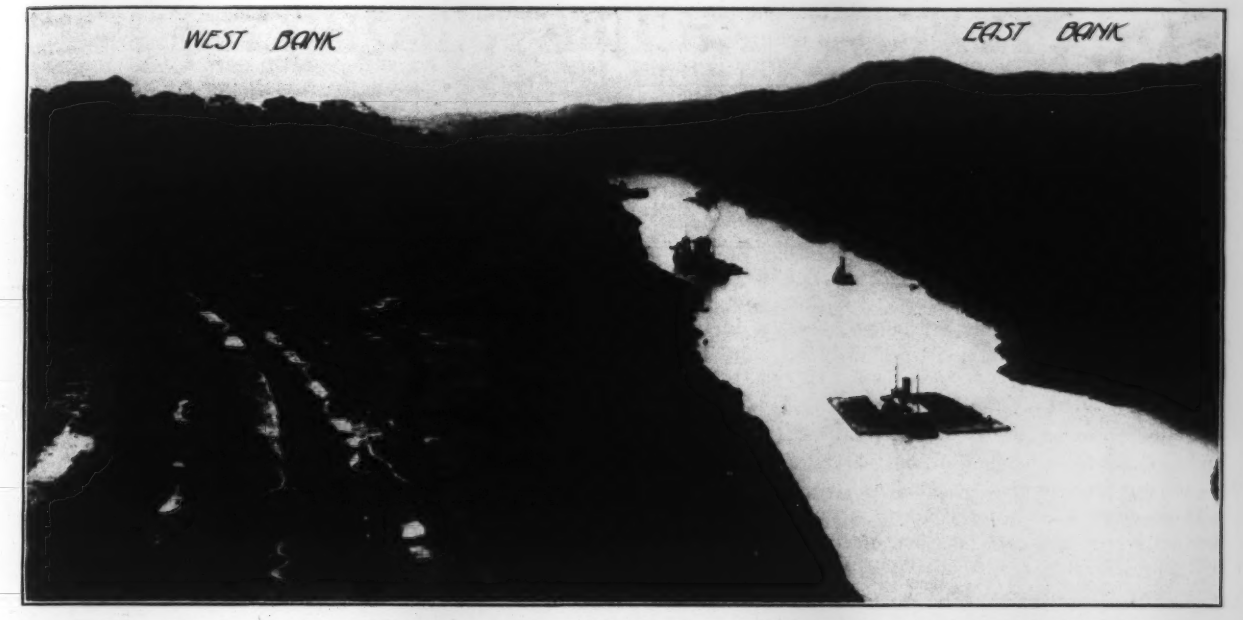


FIG. 10. BANK REDUCTION AT CULEBRA—A SLIDE-PREVENTIVE MEASURE ONLY PARTLY SUCCESSFUL

This steam-shovel terracing was done in 1910 to 1914, taking 6½ million cu.yd. from the east bank (right) and nearly 2 million from the west bank. This was prior to admission of water into the canal

sluicing plant was installed on this bank. Steam shovels and trains could not operate on any portion of the broken mass (Fig. 7).

There remained a stretch between this and the limiting crack to the east, but this ground was in such condition that before preparations could be completed for beginning work it would be broken up so that steam shovels could effect nothing. Subsequent developments have verified this.

No further cracks or indications of a further extension eastward have occurred, and judging from past experience the existing cracks will limit the fracture, which is about 1,300 ft. from the edge of the prism.

Similar investigations and considerations on the west bank show there is no possibility of handling the material to advantage by steam shovels. The configuration of the ground is such that all material in the present movement will be taken away long before the area involved could be reached. Like the east side, sluicing could only throw the material into the prism; similarly, it is believed that the limits of the break have been reached.

The conclusion reached, therefore, is that the only method is to remove the material as it comes into the cut, by the dredges. These are handling nearly 1,000,000 cu.yd. per month, at a cost of less than 30c. per cu.yd. When the break occurred in October, 1914, and the subsequent breaking up of the west bank became evident, it was decided to augment the dredging fleet by another 15-yd. dipper dredge and three 1,000-yd. barges. The latter are in service. The dredge arrived last week and will be ready for use within two weeks.

Volume of the Culebra Slide

As to the amounts involved, it is difficult to make predictions. As noted above, when Cucaracha broke loose in 1913 it was estimated that 2,000,000 cu.yd. was involved. Upward of 4,000,000 cu.yd. was taken out of the prism before the slide was entirely removed, in addition to 1,000,000 yd. washed away by the sluicing plant. In the present instance, estimating that the material must be removed from a slope extending from the bottom of the canal, reference 40, on the outward limits of the prism, up to the farthest limits of the breaks, on the theory that all the material above these planes will move into the cut, it is estimated that 7,000,000 cu. yd. will have to be removed before the slides are entirely done away with. If we assume a surface parallel to the existing one starting from El. 40, the estimated quantity is 13,000,000 cu.yd. The resulting surface will be irregular, and perhaps the more correct estimate will be between these two limits, or 10,000,000 cu.yd.

While the foregoing amounts are the estimates of what will have to be handled before all moving material is removed, it is not to be assumed that the canal will be closed to navigation for this length of time; on the contrary, all efforts are being directed toward securing a channel. The estimated amount in the canal within the limits of the prism was about 500,000 cu.yd. when the prediction was made that the canal would probably be opened by Nov. 1, 1915, and the dredges could easily handle this amount within the time limit; but in view of the wave movements and the difficulty of breaking through the central obstruction, it was not deemed wise to make any further predictions until the moving mass was in such shape that we had a channel through it which we felt reasonably sure could be maintained. To this end the dredges are operating to cut a channel wider than the full prism width, which will enable the maintenance of a channel more easily than heretofore. On Oct. 6 the length of the channel that was blocked was 1,300 ft.; this length has been reduced since then to 700 ft.; but due to the continued movement we have been unable to break through the closure in the vicinity of Sta. 1,787, which is now about 100 ft. in length. When this is accomplished the lower waves can be more easily removed and more reliable predictions made than are now possible.

The Mexican Railway; 1873 and 1915

SYNOPSIS—The location of surveys for this first railway in Mexico was made by American engineers in the face of great difficulties. Some interesting information as to these surveys, and also as to present conditions, is given in this article.

The Mexican Railway, connecting the City of Mexico with the Atlantic port of Vera Cruz, has been brought to notice in this country very conspicuously during the recent troubles in Mexico and especially during the American occupation of Vera Cruz in 1914. It was the first railway in Mexico and is an engineering work of considerable interest, partly on account of the difficulties of location and construction and partly from the fact of its having been located by American engineers. For the reasons cited it is interesting to present the following particulars of the early history of this road, condensed from information furnished by William R. Eastman, of

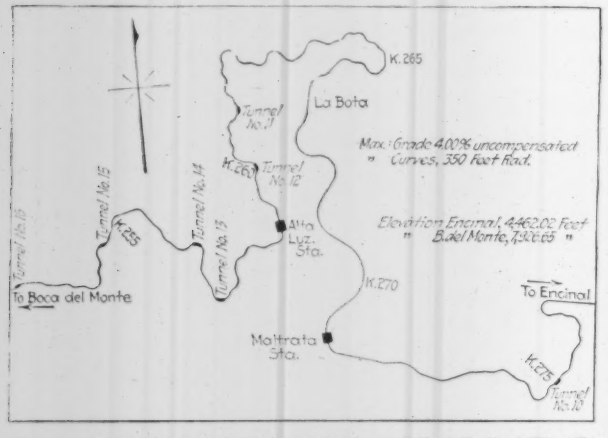


FIG. 1. LOCATION OF THE MEXICAN RY. AT MALTRATA TO MAINTAIN A GRADE OF 4 PER CENT.

Albany, N. Y., who was a transitman on the survey made in 1858 and made the difficult location at Maltrata, mentioned later. The length of the line is 264 mi.

History of the Railway

The Mexican Railway was opened Jan. 1, 1873. It had been built at great cost and represented the planning and working and vexatious delays of many years. Concessions for a railway to Vera Cruz were granted in 1837 and 1842, but notwithstanding large appropriations by the government, only 11½ mi. were built by November, 1850. The first 15 mi. extended from Vera Cruz through Tejeria to San Juan and were operated in 1858. In 1854 a line was built from Mexico to the suburb of Guadalupe (3 mi.). This was wholly of American construction. The rails and other material and the parts of the locomotive were hauled 260 mi. from the coast. In 1857 a new concession was obtained by Don Antonio Escandon, who paid $1,000,000 to the national treasury and at once entered upon the undertaking.

The broad table-land of Mexico is about 8,000 ft. above sea level. This great elevation is reached within 100 mi. from the coast, the ascent gradually growing steeper. At 95 mi. the line has risen 5,500 ft. In the next five or six miles the mountain rises 2,500 ft. higher to the Cumbres or sharp edge of the table-land, which stretches in a nearly level plain for 400 mi. To ascend this abrupt mountain wall was the railway locators' problem. There were difficulties also from deep and steep ravines, or barrancas, and a line of small hills, the Chiquihuites.

An eminent and experienced American engineer, Col. Andrew Talcott, was engaged. He was a graduate of West Point and had rendered conspicuous service in the United States Army. A surveying party of some 30 men was recruited in New York. The chief assistants were M. E. Lyons, of Reading, Penn., and R. B. Gorsuch, of New York. The latter had had experience in Mexico in the building of the experimental railway to Guadalupe; he was the first secretary of the American Society of Civil Engineers. Camp equipment, instruments, etc., were shipped from New York.

The main party left New Orleans by steamer on Jan. 1, 1858, and landed at Vera Cruz four days later. A camp was established at Tejeria, 9 mi. from the city. Three working parties were organized—one headquarters party to explore the country on horseback and two field parties. From Vera Cruz there were two main highways. One was by way of Jalapa, and had been chosen by General Scott with the American army in 1847. The other road, somewhat more direct, passed through Orizaba, climbed the mountain pass at Aculcingo and joined the Jalapa road at San Marcos; thence to Puebla and Mexico. This was the route taken by Cortez and his Spaniards. It was the regular stage route in 1858 and was adopted as the base of the survey. In many respects this was a remarkable road, built probably by the Spaniards 200 or 300 years ago.

As the disturbed political and military conditions seriously embarrassed the work, the force was reduced, keeping only one field party to examine the most critical point on the line—the abrupt ascent of the Cumbres. The road climbed the pass at Aculcingo by a series of ten or twelve extended zigzags. The railway could not do that. The attempted line of survey became hopelessly involved in a tangle of hills, and after a month's work the result was failure. Then attention was devoted to another pass, up from Maltrata, and on the farther side of which a long mountain spur thrust out for miles into the lower country. There was no carriage road in the Maltrata pass and the zigzag turns for trails were even more numerous than at Aculcingo, but hope was fixed on that prolonged spur of the mountain. After climbing to Boca del Monte, 7,922 ft., the transit was set for a down grade of 200 ft. to the mile, and with only this as a guide the mountain side was explored. No account was taken of curves, bridges or tunnels. The chopping out of the path was slow. Straight lines and their angles were carefully measured and noted. The leveling instrument was close behind.

At one point the cliff is so precipitous that it seems to hang above Maltrata, 2,000 ft. below. After winding on till the located line resembled the outline of a boot with the leg resting along the valley (Fig. 1), and after 11 mi. of sharp descent, the level of the village was reached. But this was not the end. The drainage of the valley is

through a narrow rocky gorge down which the line turns and twists until, in the valley of Encinal, it reaches level country. In 18½ mi. the descent is 3,000 ft. The average fall is 165 ft. to the mile, but many miles are on a grade of 200 ft. or more.

The crossing of the deep ravine of the Barranca de Metlac was a problem. A straight line across was only 1,400 ft., but the cost of a bridge would have been enormous and the American type of steel viaduct had not been developed. The line descends on a narrow shelf for a mile or more upstream till it reaches a broader part of the ravine and then makes an almost semicircular turn, with a bridge about 100 ft. above the water, and ascends by a similar shelf on the farther side. The river crossing at which the line reverses is shown in Fig. 2. The Chiquihuites range of hills is passed by a considerable detour away from the line of the highway.

In September, after a little more than eight months in the country, the party returned to New York, the line of survey was plotted and the report made. But for a long time nothing more was done. A new grant from the government was made in 1861. The French entered Mexico in 1862, and to facilitate their movements they contracted with Escandon to build portions of the railway. It was extended over the open country to Paso del Macho, 48 mi. from Vera Cruz. It was also carried from Mexico to Puebla. During this foreign regime 133 mi. were constructed. Still more liberal contracts were made with the republican government in 1867 and 1868.

The cost of construction was found to be unusual and extreme. The government insisted that the road should be built from both terminals simultaneously, making it necessary to haul heavy material, machinery and equipment hundreds of miles over rough mountain roads in order to build back from the capital. The work of construction of the mountain division (still under the supervision of Colonel Talcott) was taken up in earnest in 1864. But Escandon was compelled to seek outside pecuniary help, and it was an English company which (in 1872) finished the railway.

In spite of the great cost, the railway has such important traffic that good profits have been paid, at least up to the past year. The road is under English management, and the impression is common that the English built it. The fact is that the survey, the location and the plan of construction were American throughout.

Present Conditions on the Mexican Railway

Information supplementing the early history given has been furnished to *Engineering News* by W. T. Ingram, Resident Engineer. The main line and some of the branches are of standard gage, and the company owns 130 mi. of branch lines of 36-, 30- and 24-in. gage. Practically the whole of the main line and the standard-gage branches is laid with steel ties spaced 16 to the 30-ft. rail.

The rails on the mountain division are 82-lb. per yd., but are being changed to 85-lb. 40-ft. T-rails of the British standard section. On the rest of the line there are 62- to 85-lb. rails. The maintenance-of-way department is mostly under the care of Mexican roadmasters and supervisors, and all the labor is Mexican.

During 1909 and 1910 nearly $500,000 (gold) was expended in renewing bridge girders and in eliminating a number of bridges and viaducts by means of fills supported on the downhill side by massive stone retaining walls. The curious bridge at Soledad, shown in Fig. 3, has two independent steel superstructures on one set of

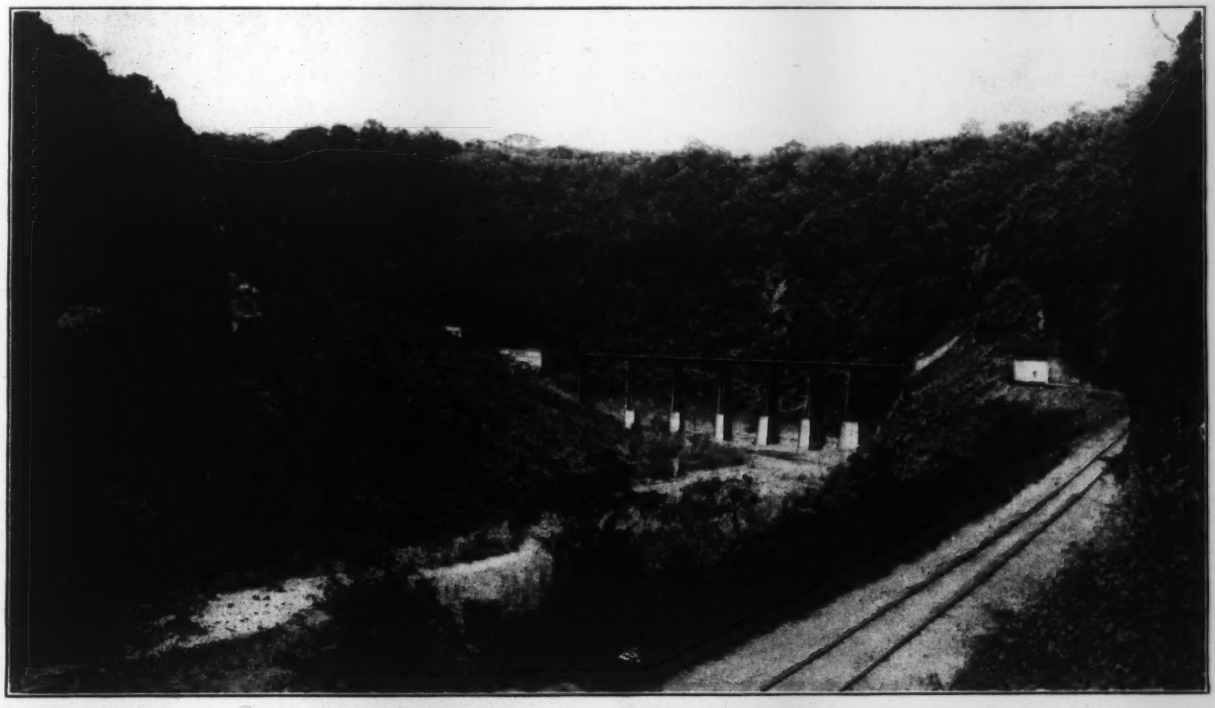


FIG. 2. DEVELOPMENT AND REVERSING CURVE AT THE CROSSING OF THE METLAC RIVER

The line on the left (entering the tunnel) descends from the high ground on a grade of 3%. It crosses the river by a bridge on a curve of 324 ft. radius, and ascends the opposite side of the valley (in the foreground) on a grade of 2½%

piers. The lower structure has through-truss spans for a roadway. The sides of the piers are carried up above the trusses and support transverse plate girders, between which are framed the plate-girder spans for the railway. Since 1908 large amounts have been spent for new rails, steel ties, ballasting, water service, fuel-oil installations, etc. The total issued capital of the company amounts to $41,557,400.

Locomotives and Operation

The operation of the 4% grade between Orizaba and Esperanza is interesting from the use of locomotives of the Fairlie duplex double-boiler type. The latest of these engines, built by the Vulcan Iron Works, England, have six 48-in. driving wheels under each end, with a wheel-base of 9 ft. 3 in. for each group and 35 ft. 6 in. for the entire engine. The weight is about 153 net tons in working order, with a tractive power of 59,134 lb. The four cylinders are 19x25 in.

The boiler is a single structure having two barrels 4 ft. 9½ in. diameter, set back to back, each with a fire-box 5 ft. 4⅛ in. by 4 ft. 6¾ in. While there are two independent fireboxes, there is no division above the crown-sheets, so that water circulates freely between the two barrels and there is an ample depth of water over the crown-sheets when the engine is on the steep grade. The train load for one engine on the 4% grade is 340 metric tons. Mr. Ingram states that these engines are very satisfactory. The whole weight is available for adhesion, and with the short rigid wheel-base they can traverse sharp curves safely and with a minimum of tire wear. The articulation of the underframes and the supports of the boilers provide for free movement longitudinally and vertically. The fuel economy is fully 20% less than for two ordinary engines of practically the same dimensions.

All freight trains are handled with two engines, one at each end. Trains of 680 tons are hauled at an average speed of 10 mi. per hr. This is considered good performance on a continuous grade of 4%, not compensated for curvature, and having curves of 350-ft. radius, many of which are reverse curves and are so short that frequently a train will be on three curves at once. Since 1909 all engines have used petroleum-oil fuel from Tuxpam. It is stored in tanks of 50,000 bbl. capacity, and

FIG. 3. DOUBLE BRIDGE AT SOLEDAD; MEXICAN RY.
The lower bridge, with truss spans, is for a highway

the service tanks for supplying the locomotives are of 500-bbl. capacity (Fig. 4).

Commenting upon the operation of the line during the present revolutionary troubles in Mexico, Mr. Ingram states as follows:

> It is very gratifying to be able to record that notwithstanding the fact that Mexico has been in the throes of a severe revolution for the past four years, no more serious damage has been done than to destroy track, burn buildings and decks of unimportant bridges, wreck engines and cars by running them wild down mountain grades, etc. While this has involved heavy outlay for repairs, it has not kept the railway out of service for long continuous periods. At least one important bridge missed destruction by a narrow margin. This is the Soledad Bridge, 27 mi. from Vera Cruz and carrying both the railway and the highway across the Jamapa River [Fig. 3]. On Apr. 22, 1914, while the United States expeditionary forces were taking possession of Vera Cruz, the bridge was mined and would undoubtedly have been entirely destroyed had any attempt been made to start up country.

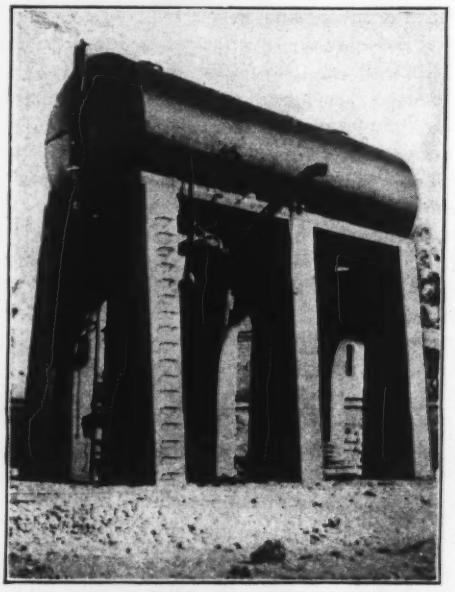


FIG. 4. OIL-FUEL STATION FOR SUPPLYING LOCOMOTIVES; MEXICAN RY.

Tests of a Proportional Weir

A proportional weir of the Rettger type (as described in *Engineering News* of June 25, 1914, p. 1409) corrected for finite length of crest was subjected to very careful tests on a large scale during the past year by E. W. Doebler and F. H. Rayfield, students at Cornell University. The hydraulic testing plant at Cornell permits of obtaining a large supply of water and of measuring discharge volume accurately by means of a calibrated standpipe 6 ft. in diameter by 60 ft. high. The methods of making the test were in all respects such as would secure a high degree of precision. Only the results will be stated here.

The Rettger type of weir, whose sides have the equation l^2h = constant, would be of infinite length at the crest if the curve were continued. For practical use the crest length must be limited to some length L, at which point the height from weir crest to curve of side is H. The roughly triangular areas outside of the side lines of the weir lost by limiting the crest length to the length L have an area equal to LH. By adding this same area LH just below the theoretical crest—in other words, moving the crest down a height H—the same total area of discharge is secured as with the theoretical Rettger weir, although the head on the small added area is slightly altered. This correction was made in the weirs tested, as indicated by the dotted line in Fig. 3.

Two weirs were tested, both 3.5 ft. long on crest, but one having $H = 0.10$ ft., the other 0.01 ft. Fig. 2 shows weir *A* discharging under 1-ft. head of water. A few feet below the weir crest the flow strikes diverters, which lead it into the measuring standpipe. Both weirs were cut out

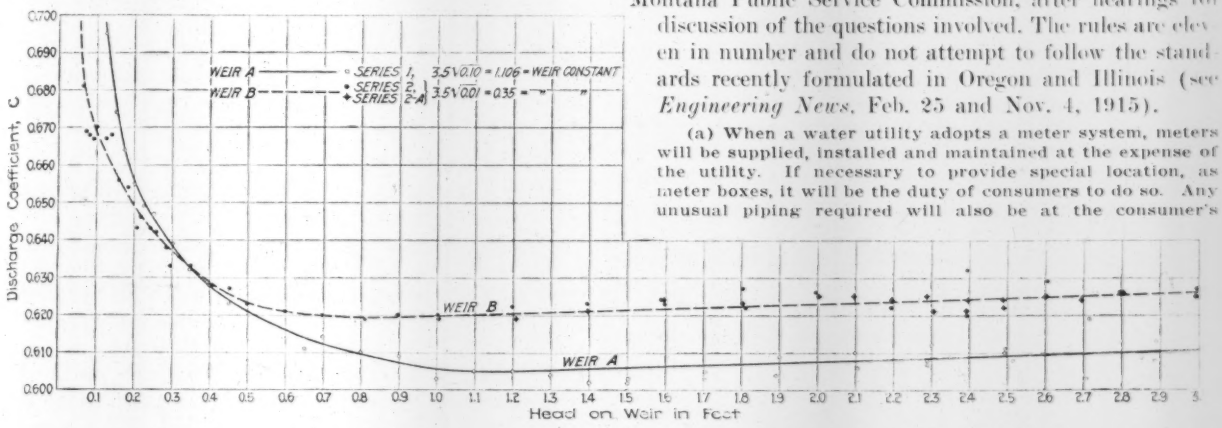


FIG. 1. DISCHARGE COEFFICIENTS FOUND BY TEST OF TWO RETTGER WEIRS

of sheets of galvanized iron and carefully handfiled down to the plotted curve, filing perpendicular to the sheet.

The results are shown by the plotted points and curves in Fig. 1, which give the computed discharge coefficients for various heads on the weir. According to the theory of the Rettger type of weir, the coefficient should be constant. For the higher head (above 1 ft.) the coefficients of the smaller weir are about 2.4% higher than for the larger weir. The gradual increase in coefficient as the head increases was found to be not due to the velocity of approach above the weir. Probably the manner of correcting for finite length of weir is responsible.

Logarithmic plotting between head and discharge gives:

$$\text{Weir A, } Q = 8.47\ h^{1.5};$$

$$\text{Weir B, } Q = 2.74\ h^{2.5}.$$

The tests noted were made under Prof. E. W. Schoder.

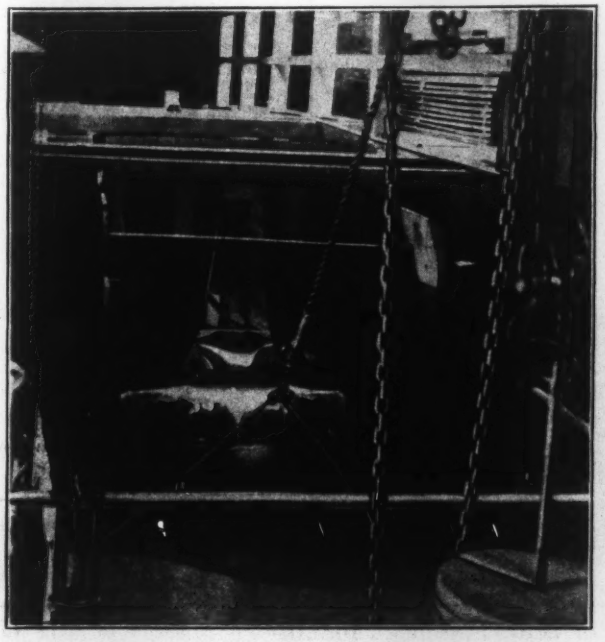


FIG. 2. LARGE WEIR (WEIR A) UNDER 1-FT. HEAD

Water-Works Rules Issued by Montana Commission

Rules and regulations governing and standardizing water-works service in Montana have been issued by the Montana Public Service Commission, after hearings for discussion of the questions involved. The rules are eleven in number and do not attempt to follow the standards recently formulated in Oregon and Illinois (see *Engineering News*, Feb. 25 and Nov. 4, 1915).

(a) When a water utility adopts a meter system, meters will be supplied, installed and maintained at the expense of the utility. If necessary to provide special location, as meter boxes, it will be the duty of consumers to do so. Any unusual piping required will also be at the consumer's expense. Meters which have been installed at consumer's expense will be taken over by the utility at the pro rata value based on length of service and present condition.

(b) If the utility requires a meter deposit, the consumer will receive interest thereon at the rate of 6% per annum.

(c) A utility may protect itself by reasonable minimum charge based on the size of the meter and consumption.

(d) The utility at its own expense must tap the main and furnish the corporation cock, the clamp (when necessary) and any other material used or labor furnished in connection with the tapping of the main. Any other material used or labor furnished in connection with the installation of the service pipe or curb box to the property line of the consumer shall be billed against the consumer at actual cost.

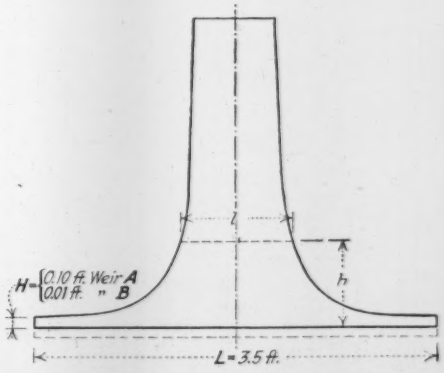


FIG. 3. SKETCH OF THE CORRECTED RETTGER WEIR

(e) Unless under conditions where it is not possible to do so, the utility must notify the consumer prior to shutting the water off.

(f) When the water has been shut off for cause, the utility shall not charge the consumer a greater sum than $1 for turning it on. This must not be indicated in the schedule as a "penalty," but is presumed to simply reimburse the utility.

(g) Should the consumer, not the owner of the property, desire service and tender the utility a deposit to guarantee payment of bills for a period not less than the customary billing period of the utility, and in no event less than one month, the utility must accept such deposit and render the service applied for.

(h) A consumer must not add additional fixtures without the consent of the utility, under flat rates.

(i) Consumers must not without the consent of the utility supply water to others.

(j) When the water has been shut off, it should not be turned on except by the utility or by its direction.

(k) The commission will not authorize fines or penalties.

Subaqueous Rock Excavation--II

By Charles C. Phelps*

SYNOPSIS—Continuation of a review of American practice in removing underwater rock. Notes on the equipment of drill barges for nontidal waters.

In a previous article (*Engineering News*, Nov. 18, 1915) the general development of the American method of excavating rock under water (drilling and shooting) was sketched and the use of drill barges for nontidal waters outlined. The notes on barge equipment are continued in these paragraphs. While the descriptions of drills and accessories and working methods apply particularly to barges for nontidal waters, yet it is obvious that this apparatus and these practices have a broader application to all subaqueous drilling.

Barge Drills and Accessories; Drilling and Charging Practices

The steels used for subaqueous work are exceptionally long, and large-diameter bits are usually employed. The distance between drill and rock is necessarily much greater than on land. With these extremely severe conditions it is necessary that equipment of the heaviest type be employed. It is particularly desirable to have the bearing surfaces of the drilling machine as liberal as possible and spaced as far apart as practical in order to counteract effectively any buckling or glancing tendencies of the drill steel. The wear on a submarine drill is very severe and for that reason the packings and bushings should be made of the best materials and be easily replaceable.

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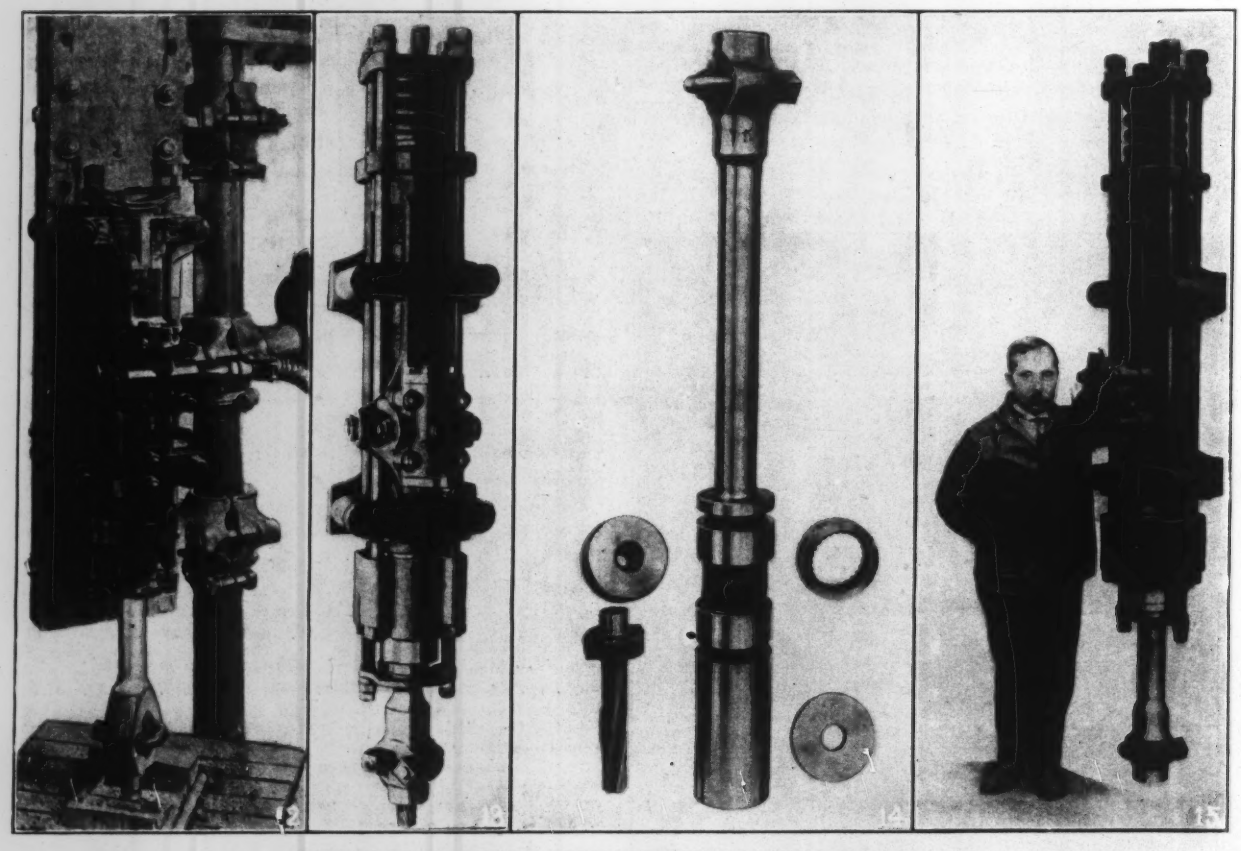
Light and Heavy Submarine Drill Types

In Fig. 12 is shown one of the lightest types of drills used for this class of work. Four of them are employed on the boat shown in Fig. 4. They have a cylinder diameter of 4¼ in. and a stroke of 8 in., and will handle steels 25 or 30 ft. in length and of about 1½-in. diameter. Their normal speed is about 400 strokes per minute. The valve action is of the standard "Sergeant" type consisting of a main pressure-thrown valve of the balanced spool type, which in turn is controlled by an auxiliary valve thrown by the piston. This type of valve automatically admits of a wide variation of stroke to meet the varying conditions imposed.

Fig. 13 shows a type which is probably in most extensive use for general submarine work. It has a cylinder diameter of 5½ in. and a piston stroke of 8 in. For extra-heavy work, where the depth of water is excessive or where larger holes must be drilled, the drill shown



FIGS. 12 TO 15. THREE TYPES OF SUBMARINE ROCK DRILLS

FIG. 12—Light design. Fig. 13—Standard weight. Fig. 14—Piston, rifle bar, rotation washer and ratchet of extra heavy drill. Fig. 15—Extra heavy drill

in Fig. 15 is also used to some extent. It has a 6½-in. cylinder and a 9-in. stroke. Both of the last-mentioned machines will handle steels 2¼ in. in diameter and 40 or 50 ft. in length. Both operate at a speed of approximately 250 strokes per minute.

Any considerable increase in the length of steel used necessitates increasing the cross-section so as to diminish the buckling tendency of a long steel. Consequently the weight of a long steel becomes very great, often as much as 400 or 500 lb. The light drill shown in Fig. 13 might be sufficiently heavy to operate steels of a given bit diameter in drilling to a moderate depth, but it might become necessary to employ a drill of the heavier type for the same diameter of hole put down to a greater depth, due to the increased power required to reciprocate the heavier steel. When the operating conditions are known it is a simple matter to find the proper type and size of drill.

Drill-Cylinder Supports

In all of these sizes the cylinder is arranged for bolting directly to a saddle (Fig. 16) which travels in the drill-frame guides. The cylinder is provided with double pads to fit the saddle, as shown by Fig. 15. These bosses are extended over the outside of the cylinder so as to be easily accessible and to insure secure bolting to the saddle.

The valve motions of the drills illustrated in Figs. 13 and 14 are exhaust-thrown, with an outside adjustment for the valve. This makes it possible to run the machine either at a long or short stroke, and the adjustment is under perfect control of the operator. The value of the variable stroke is particularly apparent when starting holes and when working in caving, seamy or broken ground, while the quick return "mucks" the hole well.

In the heavier types four through bolts connect the front-head and back-head plates. Between the back-head plate and back head is placed a heavy single-spiral spring. By tightening up the nuts on the through bolts to the desired amount, the front head and back head are made tight with the cylinder. The spring is then in compression, however, having ample reserve deflection, so that if the piston should strike the front head occasionally—either from improper attention of the operator or when it is required to back the steel out of a tight hole—the drill will not be damaged, but the hammering shock will be absorbed by the spring.

Submarine drills operating on drill barges fitted with modern equipment have given results in speed and economy of operation that compare favorably with drilling and blasting operations on land. As a matter of fact, drills of the submarine type with power-operated feeds, and mounted on self-propelled carriages, are being employed for land operations with remarkable success.

Handling the Steels

Small-size steels may be manipulated by hand, but a hoist of some kind is necessary for handling the longer and heavy pieces usually found on drill boats when removing dull steels from the chuck and inserting sharp ones. Frequently an extra sheave is mounted on top of the tower over which the rope for the drill steel passes. Cranes and overhead tracks are both employed extensively for conveying the steels to and from the smith's shop.

It is wasteful of the drill's and operator's time to stop the drill to measure the depth of the hole. A mark may be made on the steel corresponding with the depth from the surface of the water to the bottom of the hole, or the drill feed cable or rod may be marked so that the drill can be stopped without delay when the desired depth is reached. A convenient way of marking the steel is to lay off a scale of feet and inches on one guide of each drill frame, starting at the water line of the vessel, when

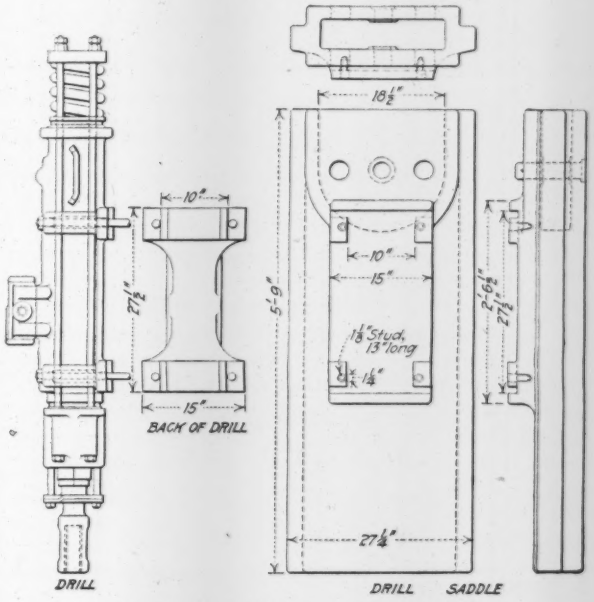


FIG. 16. SUBMARINE-DRILL SADDLE

raised on the spud anchors, and running upward. With the bit touching the surface of the water, the desired depth of cut can then be marked directly on the steel.

Power Supply for Submarine Work

While submarine-drilling work has been performed with both steam and compressed-air equipments, the former have advantages that greatly outweigh the benefits of operating by means of air. As a result, steam is almost universally used. The steam pressures employed generally range between 90 and 100 lb.

In selecting a boiler for this service, full allowance must be made, in addition to that consumed by the drills, for steam for the auxiliary apparatus, such as pumps, hoists, dynamo engines for lighting service, etc. Reduced efficiency and capacity, due to soot or scale in the boiler after continued use, must be foreseen. In determining upon the type and size of boilers for submarine drilling it must be remembered that the service is almost continuous and there is very little opportunity to clean the boilers.

In locations where the cost of boiler water or coal is excessively high it might pay to exhaust the drills to condensers, although it would probably not be worth while under ordinary conditions. One notable instance where a condenser was employed to advantage was in the drilling of a channel in St. Georges Harbor, Bermuda, during 1913 and 1914. Practically all the fresh water in Bermuda is rain water caught and stored in tanks and sold at various prices up to $4.50 per ton. The condenser in this case consisted of 160 ft. of 2-in. galvanized-iron pipe bolted under the bottom of the scow. Even in the summer, when the temperature of the sea water was about 75° F. a vacuum of 22 in. was easily maintained. The two

Type H-9 Ingersoll-Rand drills and the auxiliary equipment were operated with steam from a boiler carrying 60 lb. gage pressure, the condenser increasing the effective pressure to about 70 lb. The water consumption amounted to only about 1½ tons per week for each drill and its auxiliary apparatus. In addition to the water saving and reduction in coal consumption, the noise and inconvenience of the exhaust steam were eliminated and pure boiler-feed water was assured. Practically no cylinder oil was used for internal lubrication, so there was no trouble from oil accumulating in the boiler.

Drilling and Charging Aids; the Submarine Tube

In almost all subaqueous operations the drilling is done through any overburden that may be deposited on top of the rock. If this overburden is not prevented from getting into the drill hole, the drilling progress is liable to be very slow, or perhaps even entirely interrupted. Likewise, in drilling a range of holes where the longitudinal axis of the boat is more or less parallel with the direction of flow of a swift current, the débris that is washed out of the upstream holes by the jet or blown loose by blasts may drift down upon the drills which are working farther down stream, clogging them and often increasing the time necessary for drilling, cleaning and charging. To overcome these difficulties the submarine drilling and charging tube has been devised. The drill is run to the top of the guide and a protecting guide tube is lowered and driven, with the assistance of a water jet, down through the overburden to the rock. One of the commonest forms of drilling tube is shown in Fig. 17. It consists of a telescopic tube of large diameter. To the bottom end is connected an extension pipe slightly larger than the drill hole required. The extension pipe sinks down to and rests upon the solid rock and the drill steel passes through the tubes. Near the bottom of the tube, but far enough up to clear the overburden, a "cross" connects the large telescopic pipe and extension pipe. Sometimes a tee is used instead of a cross. This cross gives an opening on either side of the pipe, through which the cuttings are discharged. The telescoping construction accommodates different depths of water, as the top of the tube is generally held a little above the surface of the water. Where this tube is used in connection with barge or submarine-frame drilling the top of the tube may be bolted to a platform overhanging the side of the boat or to a crosshead arranged to travel vertically so that the drill steel will enter it easily and be in line with it. When work at a hole has been completed, the lower section of the tube can be raised by the chain so as to clear the rock prior to setting off the charge.

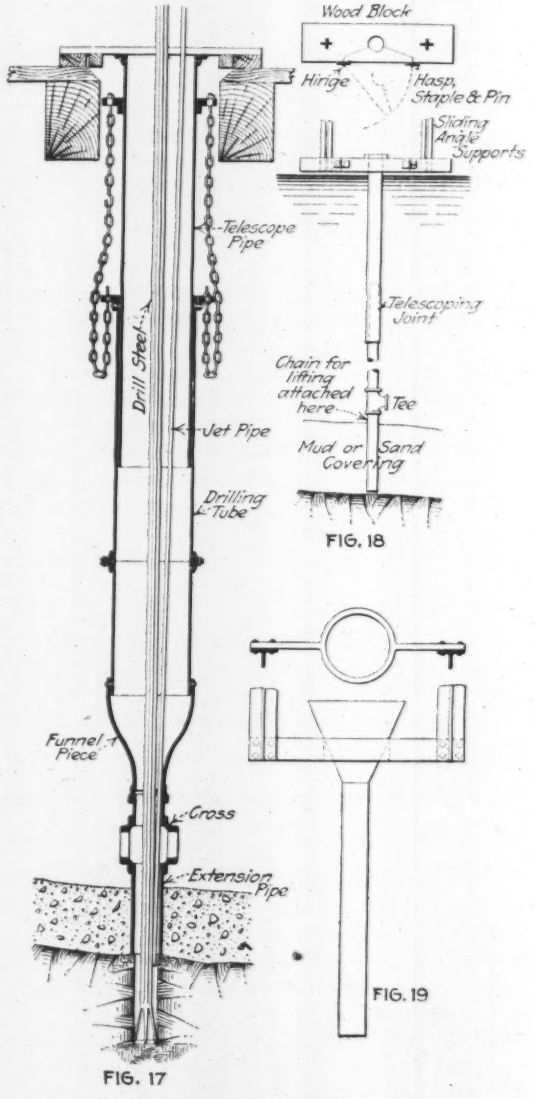
These submarine tubes are also made in other forms. Fig. 18 shows an arrangement in which the tube is bolted to a crosshead which slides in guides, so that the apparatus can easily be raised and lowered by the hoisting engine, or by hand if counterweighted.

Where the depth of water is nearly uniform the telescopic construction is not as necessary, in which case a stiff tube, built up of any desired number of sections, may be used without serious difficulty.

A similar device that works satisfactorily in locations where the water is not too deep and the layer of overburden not too thick is known as a sand pipe (Fig. 19). It is simply a large cast-iron funnel, the one shown being about 5 ft. in length and large enough to accommodate the charging stick. The sand pipe rests in position in a cast-iron support of the shape shown in Fig. 19, which rests on top of the overburden. The ends of this casting are riveted to the ends of two T-bars that slide in guides or are otherwise arranged for raising and lowering.

The sand pipe is allowed to slide down into the overburden before a hole is started. It works its way to the bottom by its own weight. Like the drilling tube, it prevents sand from getting into the hole when it has once been emptied of sand by the washout jet.

The sand pipe and its support are raised and lowered by cables, usually counterweighted. If the sand layer is deeper than the spigot of the funnel, the end of the sand pipe will of course not rest on the rock. This may add to the difficulty of keeping the hole clean. However, the device will aid in this respect, and in any event will serve perfectly as a guide for the steel and the charging stick, so that the hole is kept relatively clean.



FIGS. 17 TO 19. SUBMARINE DRILLING AND CHARGING TUBES AND ACCESSORIES

Fig. 17—Telescopic tube. Fig. 18—Tube and crosshead in guides. Fig. 19—Sand tube

After a bit has once started a hole, the complete drilling is generally accomplished without interruption. If there were no sand pipe or drill tube, as soon as the steel was raised sand and small stones would fall into the hole and cause trouble and delay. These devices are of great assistance in charging, as it is very easy to locate the large funnel opening. Without some form of guide the charging is often very difficult, especially in swift currents, because the charger, in moving around the edge of the drilled hole, often causes a slide of sand and stones into the hole, necessitating redrilling.

In all subaqueous drilling operations it is necessary to get rid of the cuttings from the bottom of the drill hole, so that the steel will not bind and every blow be delivered against clean, solid rock. In most cases this is accomplished by means of a jet pipe placed in the hole alongside the drill steel and kept far enough from the bottom so that the bit part of the steel will not touch it on the up stroke. The jet pipe is handled from the deck of the barge or platform by means of the overhead rigging when the size of the equipment warrants it, but in shallow water and for the smaller holes it is more usual to manipulate it by hand. Various sizes of jets have been employed, but probably a ⅜- or ½-in. pipe is most suitable for the purpose, and a stream of water, usually under 100 to 150 lb. pressure, is forced through the jet pipe, effectively washing out the cuttings from the bottom of the hole.

Besides its main function, the jet serves the very useful purpose of assisting the submarine drilling tube, or sand pipe, to settle rapidly through the overburden and down to the rock bottom.

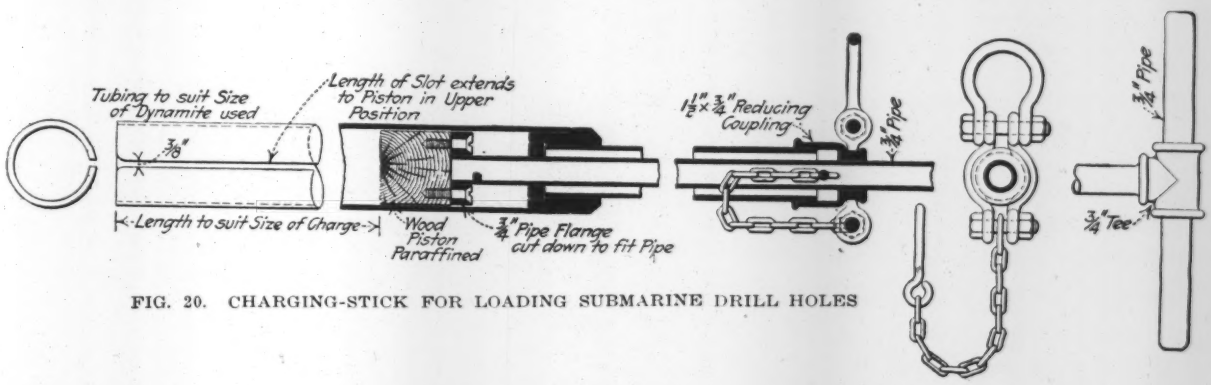

Tubing to suit Size of Dynamite used
Length of Slot extends to Piston in Upper Position
⅜"
Length to suit Size of Charge
Wood Piston Paraffined
¾" Pipe Flange cut down to fit Pipe
1½" × ¾" Reducing Coupling
¾" Pipe
¾" Pipe
¾" Tee

FIG. 20. CHARGING-STICK FOR LOADING SUBMARINE DRILL HOLES

Loading the Drilled Hole

The loading of the holes is done most conveniently by means of a loading tube, also known as a loading stick, illustrated in Fig. 20. The bottom of the loading tube has a slightly smaller outside diameter than the hole drilled and serves as an envelope for the dynamite sticks. This large tube or envelope for the dynamite sticks is manipulated by means of a small pipe inserted in the upper end. The complete length of pipe depends on the depth of water and depth of hole drilled. Through this tube passes a still smaller pipe or rod and extends into the larger pipe at the bottom. To the end of this last rod is attached a wooden piston which fits fairly tight into the large tube. The charge is inserted in the cavity in the large tube, with the plunger up, and the pin is slipped into the hole provided for that purpose in the small rod.

The blaster inserts one stick of dynamite after another in the charging tube, shoving them up into place until the pipe will accommodate only one more stick. This stick has a piece of wire or rope around it or a small wedge is inserted to keep the sticks from falling. The loaded charger is lowered into the cleaned hole with the aid of the hoist and the hole charged. In deep holes the above foregoing operation may have to be repeated one or more times.

The fuse wires are carried through the slot cut in the large bottom tube. The loaded charging stick is then inserted in the drilled hole and lowered to the bottom. The advantage of the drilling tube or sand pipe for rapidly guiding the loading stick into the hole will now be more apparent. When the stick is in the correct position, the pin is released, the inner rod is pressed downward and at the same time the outer tube pulled up, leaving the charge in the hole with the fuse wires ready for exploding. When the required number of holes have been drilled and charged the wires are connected to the electric battery or firing machine and the charge is then ready to be set off.

The charging tube is manipulated by hand, usually with the assistance of block and tackle. Instead of the rod and wooden piston previously mentioned, an older method employs a long wooden pole, which is used in ramming the powder out of the charger and into the hole.

On many boats a signaling system is used in connection with charging. When a hole has been completed, the drill runner rings a bell or blows a whistle the proper number of times to notify the blaster which frame is ready to load. A steam whistle attached to each drill feed is convenient on a large boat.

The holes may be shot singly, in single rows or in a battery of rows. These methods are known as "single-hole," "range" and "field" blasting respectively. There are advantages in each method. However, it is generally believed that the breaking effect is much weaker when the holes are shot separately. Nevertheless, in cold-weather operations single-hole blasting is employed almost exclusively, as any delay would chill the dynamite and so weaken its action, if not actually prevent its detonation. This method also has the advantage of greater simplicity, as the holes are usually shot as soon as drilling is completed. In field blasting sometimes only a few of the holes, say every fourth or fifth hole, are exploded di-

rectly by the battery, the detonators and powder charges in the other holes being set off by the shock of these explosions. Nitrogelatin is more suitable for subaqueous work than nitroglycerin, as the water has a tendency to replace the latter substance in dynamite.

The inexperienced operator should use every caution to be at a safe distance when setting off the charge. However, many operators consider it safe to set off the charge without removing the drill boat or platform if the water is not too shallow, if too many holes are not set off simultaneously and if the individual charges are not too heavy. Such a procedure should never be attempted, however, until experience has taught the range of safety, as otherwise a broken spud or even more serious damage may result.

When the barge must be moved prior to blasting, the spuds are drawn up and the barge is shifted by winding in and paying out on opposite anchor ropes until it is clear of the charged holes. After blasting, the barge is brought back by means of the winches and a new series of holes is drilled.

Another method of working across a wide channel is to swing the boat on one spud by means of the current after the completion of each range, working diagonally across the channel from range to range.

One method of measuring the distance from range to range, so as quickly to locate the boat in each new position, is by means of wires stretched from the ends of the boat to two poles, either inserted in drilled holes in the channel or erected on shore. Surveyors' transits mounted on the shore are also sometimes employed in locating the ranges.

Sharpening Steels for Subaqueous Work

Where the sizes of bits do not exceed 3¾ in. it will often pay to install a sharpening machine; however, for the larger-size bits generally used for subaqueous work it is more usual to employ hand methods of sharpening. As the efficiency of the cutting bit greatly influences the speed and cost of the drilling operations, it is advisable to procure the services of a skilled blacksmith to do this work.

In dressing the bit care must be taken not to overheat it or to work while too hot. The proper temper should be determined by experiment and rigidly adhered to. After forging the bit should be allowed to cool thoroughly and then be reheated for tempering. It should never be sharpened and tempered on the same heat. Plenty of metal should be left back of the cutting edges, not only to provide ample strength but also to maintain the gage for as deep a cut as possible. Whatever style of bit is adopted, it is of the greatest importance that it be made as nearly symmetrical as possible.

Removing the Shattered Rock

Rock that has been coarsely broken is most economically dredged by the elevator type or the dipper type of dredger. Where the rock is well shattered and where the depth of each breaking is limited to 2 or 3 ft., as is usually the case with rock breakers, the elevator dredger, because of its ability to dredge closer to a given grade, is more effective. Where the rock is broken in large pieces as usually occurs with the drilling and blasting method, and where the depth of the cut is more than 3 ft., the dipper dredge will demonstrate superior economy. This does not apply to excavations in depths of 35 ft. or more, as the dipper dredgers, owing to limitations of mechanical design, lose their effectiveness beyond certain depths. (See General Report, 12th International Congress of Navigation, Philadelphia, 1912, on the subject of "High Powered Dredgers and Means of Removing Rock Under Water.")

Modern powerful dipper dredges carry buckets of from 5- to 15-cu.yd. capacity. The dredge is maneuvered so that the face of the excavation is always in front of the bucket, which is operated from the bottom of the face upward.

Dipper Dredges Cut Cost of Drilling

When the majority of rock to be removed lies in large masses, as in extensive channel improvements, the bottoms of all the drill holes are kept approximately on the same level. When these holes are charged with dynamite and exploded, the rock at a certain level above the bottom of the hole will be broken into small pieces, and above this elevation the pieces will be larger. The dredge bucket enters the mass of broken rock at the level where the rock is most shattered and is hoisted up under the larger and heavier pieces. Ability of the dredge to handle large masses permits of spacing the drill holes farther apart, thus lowering the average feet of drilling per cubic yard of excavation and also the unit amount of explosive.

Elevator or ladder dredges, owing to the small size of their buckets, require the rock to be broken into smaller pieces. If the pieces are too large they are rolled in front of or fall behind the buckets or are forced to the side of the channel. To remove these large stones entails the extra cost of divers or the employment of a grab dredger.

The grab, orange-peel or clamshell dredgers have not proved economical in removing large masses of rock for the reason that the surface of the rock must be thoroughly broken up to permit the bucket edges to penetrate. To obtain this condition entails the use of unnecessarily great quantities of explosive. Even where the rock is well broken there is still great uncertainty, as the amount lifted each time the grab is hoisted is dependent on the chance of dropping on material arranged in such form that the grab may catch a good load. The grab has not the positive manner of working of either the dipper or elevator dredger.

It is on small, disconnected ledges of rock that the grab dredge, because of its comparatively small cost of construction, may be employed to advantage.

Electrification of Railway Terminals in Chicago is impracticable for financial reasons, according to advance statements regarding the comprehensive work of the Chicago Terminal Electrification Commission on electrification and smoke abatement. The cost is estimated at $290,000,000 for 3,356 mi. of track which would have to be included in a complete system. This is for the terminal system as a whole, with electric service extended to points 15 to 20 mi. distant. Of this amount $188,000,000 is said to be the direct cost and $102,000,000 the cost of changes and betterments in connection with the work. The committee has not considered the practicability of electrification for individual roads. As far as the smoke problem is concerned, railway locomotives are said to be a minor factor, and a permanent commission is proposed to study this problem. One conclusion reached is that the complete elimination of steam locomotives is not necessary as a means of smoke abatement. A second finding is that no existing contact system is well adapted to some of the Chicago trackage. A third point is that the longer electrification is delayed the more the technical difficulties will be reduced through developments of the art.

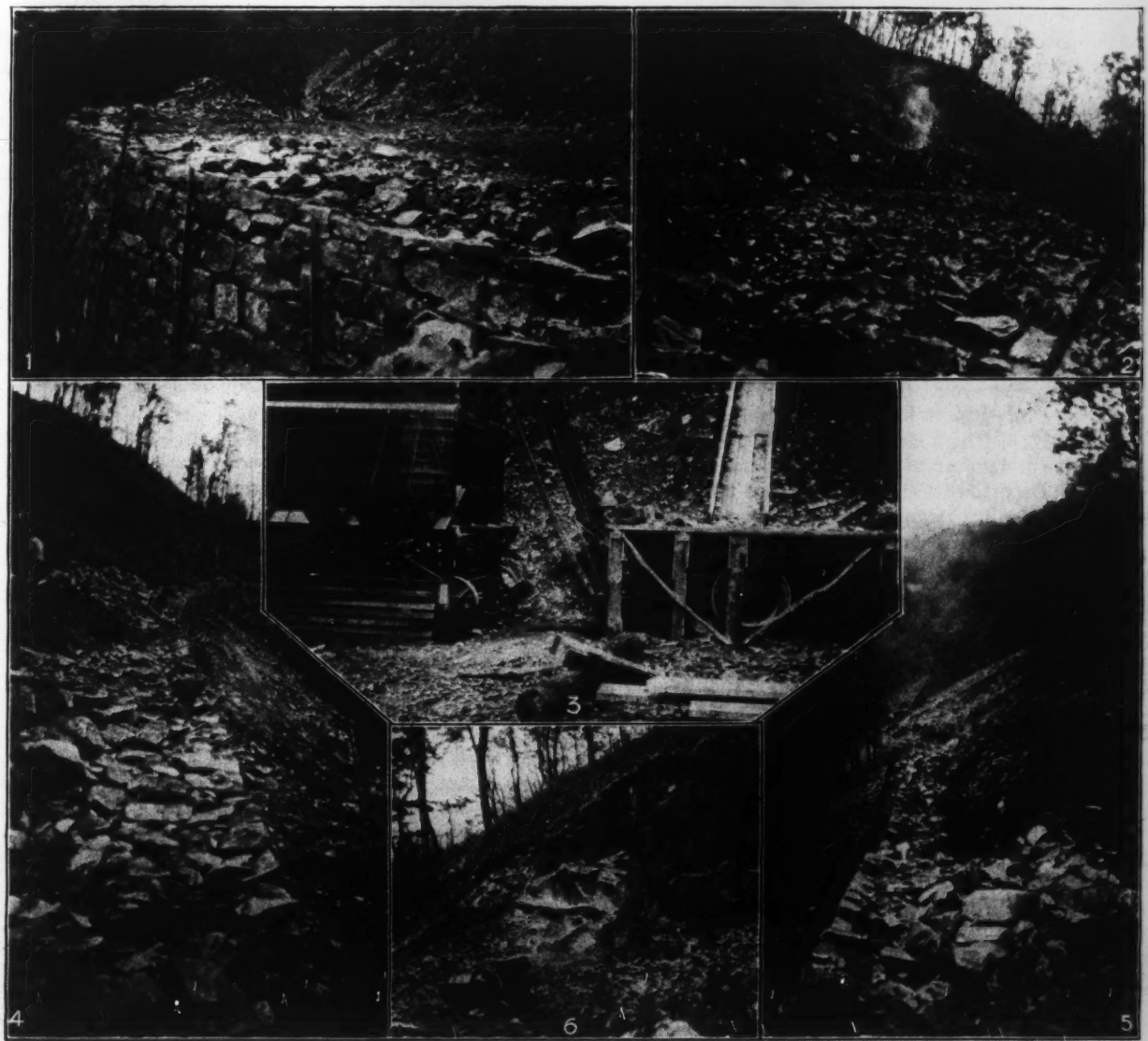
Alpine Roadbuilding in the Hudson River Valley

Northwest of the town of Nyack, Rockland County, N. Y., a 3-mi. section of state road is under construction amid such picturesque surroundings that photographs of the mules working in paths on the sides of the hills have been exhibited as pseudo-Alpine scenes. The road here is half or three-quarters of a mile back from the Hudson River, gradually climbing up the face of the hills which border the river valley. The road cuts through this divide at about the middle point of this contract section—a very happy arrangement, as will be shown.

The side-hill slope averages probably about 45°, but is steeper in many places. It is about the angle of repose for a great mass of traprock fragments that cover the hill sides, while the hills themselves are largely composed of solid trap, with a rather light surfacing of earth. The old road follows the foot of this steep slope to a natural gulch, where a "hairpin" curve, so called, practically doubles the road back on itself—what would be a switchback in railway terms.

The plans and cross-sections for this stretch of highway made the roadbuilding look like a fairly rugged proposition; and to pass along the old road and look up at the survey center line on the mountain side probably did not detract from the preconceived ideas that the road was one of extreme difficulty to build. The estimated quantities in round numbers were 12,500 cu.yd. of earth excavation, 5,000 cu.yd. of rock excavation, 1,500 cu.yd. of dry-laid walls and 250 cu.yd. of mortar-laid masonry. Bidders were comparatively few and prices were high.

The successful bidder was the Brady-Oltarsh Construction Co., New York City, and the contract prices were: Earth excavation, 40c. per cu.yd.; rock excavation, $1.50 per cu.yd.; dry wall masonry, $4 per cu.yd.; telford base for pavement (in place), $1.45 per cu.yd.; foundation course for a penetration bituminous-macadam pavement, $2.50 per cu.yd.; surfacing stone, $2.75 per cu.yd. The time allowed for completion is 140 working days.



FIGS. 1 TO 6. BUILDING A MOUNTAIN ROAD THROUGH A SECTION OF THE HUDSON RIVER VALLEY, NEW YORK

So much ingenuity has been used by the contractors that instead of an extremely difficult job it turns out to be one of comparative simplicity. Work was begun at the "hump" in the middle of the contract. The object of this is to have no uphill hauling. A mule path was first dug along the mountain side, on the outer line of the projected road, and the dry-masonry retaining wall started. The stones for the walls were within easy reach, as the views show. The wall building and excavation were carried on alternately. The wall was built up a little and then the fill was widened a little by excavating into the mountain side. In some places it was merely necessary to start a small slide from above and the fill made itself. Meantime some of the rock was hauled back to the beginning of the contract section to make the telford foundation.

At a convenient place on the old road, about 25 or 30 ft. (vertically) below the new location, a portable stone-crushing plant has been erected (Fig. 3). A wooden chute leads to the crusher platform from the edge of the retaining wall above. The stone excavated from the road bed or picked off the mountain side is brought to the mouth of the chute, sent through the crusher and hauled to the pavement by 7-ton motor trucks. The whole lay-out is a study in conservation of material, equipment and effort.

A steam road roller is about the only heavy equipment on the job. Single-mule stone boats are used for handling the heavy stones to wall or crusher. A small portable gasoline-engine air compressor operating a small hammer drill (Fig. 6) is being used for rock excavation. This has been found by experience on this job to do the work of four hand drillers.

The roadbed construction is substantially the same as that designed for the Storm King road, described in *Engineering News* of Oct. 14, 1915, p. 724. The retaining walls are dry masonry with a 5-ft. top wall of masonry laid in cement mortar. The dry walls are laid with a face batter of 1 on 6, and a base width of one-half the height. The road is officially known as County Highway No. 1295. The plans and specifications were prepared by the State Highway Department. H. Eltinge Breed is First Deputy Commissioner in charge of design and construction and B. H. Waite is Division Engineer.

Is a Part Stronger Than the Whole?

By R. Fleming*

The object of this article is to throw light on the question whether a structure or a member may sometimes be strengthened by cutting away part of it—a question which in some form often confronts the structural engineer.

Case 1—In a group of buildings now being finished there are many eccentric connections. The detail shown by Fig. 1 is used several times.

According to accepted rules the shear on the lower rivet is 3,000 lb. due to the direct pull of the rod, plus 1,800 lb. due to rotation about the center of gravity of the four resisting rivets. This equals 4,800 lb., which exceeds the allowable shear of 4,000 lb. on a rivet. If the plate were cut along the dotted line or the upper rivet knocked out, there would be no tendency to rotation and the shear on the remaining three rivets would be 4,000 lb. each. Shall the erector go over the work and knock out the upper rivet? In this particular detail it could be done, but in other places, where the connection is made eccentric to take a strut, it would be difficult to make a change. What shall be done?

Case 2—This case is largely hypothetical, but well illustrates the point. The architect's design of a building called for some 12x12-in. wooden columns 20 ft. long resting on a 24x24-in. cast-iron base. According to the formula used the columns were good for a load of 115,200 lb., equivalent to 800 lb. per sq.in. of cross-section. The contractor happened to have on hand some 12x16-in. timbers and used them as shown in Fig. 2.

For such a column, according to the Rankine formula for eccentric loading, the stress varies from 150 to 1,050 lb. per sq.in. If the timber were sawed along the dotted line, there would be no bending and the allowable stress

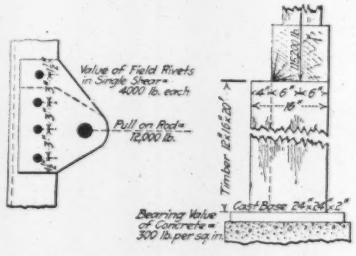


Fig. 1 Fig. 2 Fig. 3

Fig. 1—Eccentric connection. Are three rivets stronger than four? Fig. 2—Post on eccentric base-block. Fig. 3—The triangular beam—Emerson's paradox. Reproduction of Emerson's Fig. 15, Pl. XI

of 800 lb. per sq.in. would not be exceeded. Shall this sawing be done?

Case 3—Three years ago an engineer designed the foundations for a 36-story building in New York City. The footings consisted of distributing girders resting on a concrete caisson 6 ft. wide, about 50 ft. deep, running through about 10 ft. of hardpan to rock. The column center was 2 ft. from one edge of the caisson, thus making the load eccentric. For this reason the design was condemned. Had the caisson been 4 ft. wide, there would have been no criticism. But a pneumatic caisson cannot be built 4 ft. wide. At least 6 ft. is required to give a practical working-chamber underneath. The question was purely whether a 4-ft. wall was weakened by adding 2 ft. to one side. The two engineers that had final authority in the matter decided it was. Were they correct in their decision?

Johnson concludes the article, "Weakening Effects of Eccentric Loading," in his book, "The Materials of Construction," with: "Hence we may say that the addition of material on one side of a member subjected to a direct stress symmetrically placed weakens it until the added material has exceeded twice the original thickness of the member, the maximum weakening occurring when the added material is one-half the original thickness, when the enlarged member is only three-fourths as strong as the original member." The mathematics of the article is simple and perfectly correct, but is the conclusion always true?

In the *Proceedings of the Engineers' Club of Philadelphia* for October, 1891, there is a communication from Prof H. W. Spangler, "Rupture Under Tension Unsymmetrically Applied," in which he quotes a letter from the president of the club: "The original investigation of the

*American Bridge Co., 30 Church St., New York City.

strength of a flat bar of increasing width to one side of the line of strain and the discovery that the weakest section in such a bar is 1.5 times the width of the section subject to uniform stress should be credited to Mr. Carl G. Barth. . . . It was surprising to be told that as the bar widened, it at first grew weaker and then stronger . . . but an investigation of the problem showed that it yielded readily to the following analysis, which confirms the statement and shows how easily structures may be weakened by an abundance of badly disposed material."

The *Home Study Magazine*[1] for September, 1897, has an article, "Eccentric Loads—Some Surprising Facts—Gaining Strength by Removing Material," by Carl G. Barth, closing with: "We now draw the rather astonishing conclusion that more or less strength is always gained by the removal of the eccentric portion of an eccentrically loaded rectangular section, so long as such removal involves less than two-thirds of the total section."

The idea of a part being stronger than the whole is by no means new. Emerson's "The Principles of Mechanics"[2] was regarded as an authority in England 150 years ago. Near the close of the section on "The strength of beams in timber" are the few lines stating what is known as "Emerson's paradox." They read: "Let *AE* (Fig. 15, Pl. XI) be a beam in form of a triangular prism, and if $AD = {}^1/_9\ AB$, and $AI = {}^1/_9\ AC$, and the part *ADIF* be cut away parallel to the base, the remaining beam *DICEF* will bear a greater weight *P*, than the whole *ABCEG*, or the part will be stronger than the whole, which is a paradox in mechanics." See Fig. 3.

Emerson gives no proof of his statement, but follows it with the sentence: "And upon the same principles an infinite number of questions of like kind may be resolved, which are curious enough and of great use in the common affairs of life."

In the *Philosophical Transactions of the Royal Society of London* for the year 1818 there is a paper, "Account of Experiments on the Strength of Materials. By George Rennie, Jun., Esq." A portion records "experiments made on the transverse strain of cast bars, the ends loose, June 8, 1817." An interesting extract is: "The paradoxical experiment of Emerson was tried, which states that by cutting off a portion of an equilateral triangle (see page 114 of Emerson's 'Mechanics') the bar is stronger than before, that is a part stronger than the whole. The ends were loose at 2 ft. 8 in. apart as before. The edge from which the part was intercepted was lowermost, the weight was applied on the part above, it broke with 1,129 lb., whereas in the other case it bore only 840 lb."

A name well-known in engineering science is that of Hodgkinson. The *Report of the Seventh Meeting of the British Association for the Advancement of Science* (held at Liverpool, September, 1837), being Vol. VI of the British Association Reports, includes a paper, "On the relative Strength and other mechanical Properties of Cast Iron obtained by Hot and Cold Blast. By Eaton Hodgkinson, Esq." Experiments 8 and 9 for Transverse Strength were with cast bars, the section of which was an isosceles triangle, the base being 1.43 in. and each side 2 in. Experiment 10 was with a bar same as in 8 and 9 but with one-tenth the depth taken off the vertex. In all cases the bars were cast 5 ft. long and were supported on props 4 ft. 6 in. apart. The vertex was downward. Hodgkinson writes: "The breaking weights of the whole triangle, in the hot blast were 672 and 812 lb., mean 742 lb. and of the frustum 782 lb. In the cold-blast iron the whole triangle was broken with 815 lb. and the frustum with 677. The difference in the transverse strength of the hot- and cold-blast Carron irons, No. 2, is very small, the ratio between them being 99 to 100. We may therefore assume their strengths to be the same, and taking an arithmetical mean between all the strengths we have strength of triangle 766 lb., strength of frustum 702 lb. The frustum is therefore weaker than the triangle."

Coming now to Saint-Venant, "the foremost of modern elasticians," we find that his "Résumé des Leçons . . . par Navier," Paris, 1864, devotes 100 pages to "Rupture par Flexion." Professor Pearson thus comments on this portion of the book:[3] "This practically deals with the formula for the maximum moment, $M_0 = \frac{T_0\, w\, K^2}{h}$ . . . Saint-Venant holds that if T_0 be the stress at which enervation commences, we have in reality a condition for the safety of a permanent structure. . . . On pp. 95-101 our author treats of 'Emerson's paradox' or the existence of 'useless fibers.' In other words, the expression $\frac{w K^2}{h}$ can be occasionally increased by cutting away projecting portions of w. We have the cases of beams of square, triangular and circular cross-sections fully treated, as well as that of the *croix d'equerre*. The elastic failure of such outer fibers does not, however, denote that the truncated section possesses greater strength than the complete section as Emerson argued from the formula, Rennie confirmed and Hodgkinson refuted by experiment. Saint-Venant very aptly terms them *fibres inutiles*."

Quoting once more from Professor Pearson[4] (referring to the Hodgkinson experiments): "Here it seems to me that theoretically the triangle would certainly show rupture at a less load than the frustum, but it does not follow because the vertex of the triangular prism was ruptured that the triangle itself would break at a *less* load than the frustum." It is clear that in theory a crack should not extend down into the part that would stand the stress alone. Some think[5] that in actual practice if a crack ever started it would continue past the critical point and sever the beam.

In conclusion, the writer recognizes with Saint-Venant and Professor Pearson a distinction between rupture and strength. The fallacy in Barth's analysis of the tension in a flat bar with an eccentric pull is the premise, "Since the bar is only as strong as the outside fibers. . . ." Returning to the three instances cited in the beginning of this article: It is not believed that the wooden column in Case 2 will be strengthened by cutting off a part of it, or that the concrete caisson in Case 3 was weakened by adding 2 ft. to one side. And Case 1 in which he has been particularly interested remains as it was detailed and erected—no field rivets were knocked out.

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[1] The "Home Study Magazine" was one of the many predecessors of "Power." It was published from 1896 to 1898.

[2] William Emerson (1701-1782) was the author of a number of mathematical treatises. The first edition of the "Mechanics" was published in 1754, the second in 1758, and many subsequent editions were called for. The writer has seen London editions bearing dates as late as 1827 and 1836.

[3] "A History of the Theory of Elasticity and of the Strength of Materials." Todhunter and Pearson, Vol. II, page 114.

[4] Todhunter and Pearson, Vol. I, page 515.

[5] "Engineering News," Vol. 62, page 439 (Oct. 21, 1909).

Connecticut Shipping Terminal at New London

By W. E. Clarke*

SYNOPSIS—A 1,000-ft. pier, 200 ft. wide, being built by the State of Connecticut at a cost of $1,000,000. Article outlines the alternative designs which were prepared and describes the design now being executed, which is a fill inside of masonry bulkheads, with outside deck made up of precast concrete slabs resting on creosoted timber piles. Specially designed for a commercial life of 25 years.

In 1911 the Legislature of the State of Connecticut appropriated $1,000,000 to be used for the construction of a steamship terminal at New London. Field operations were begun in August, 1912; wash borings were taken, and a detailed survey was made to determine the material under the foundation of the structure.

It was understood, first, that a complete unit of a steamship terminal should be constructed with the $1,000,000 available; that it should consist of a pier, shed, warehouse, track connections and a storage yard; that these factors should be so balanced and equipped that all classes of freight might be handled; secondly, that the type of construction adopted should be such that the money expended would earn a maximum rate of interest as a direct investment.

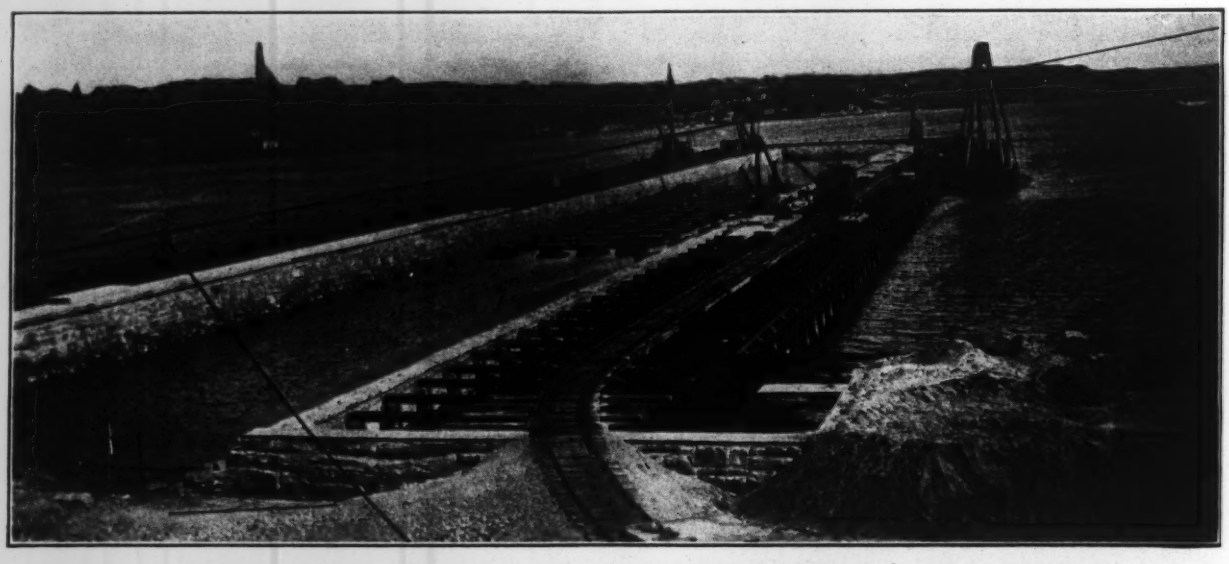


FIG. 1. RECENT VIEW OF NEW SHIPPING PIER UNDER CONSTRUCTION AT NEW LONDON, CONN.

Commercial Life of Terminal Unit

The commercial life of the unit was assumed to be 25 yr. This assumption was based on the belief that even though the unit was built to meet the maximum requirements of the present time, 25 yr. would so change the equipment used in transportation that an entirely new structure would be necessary. This period is the commercial life of the unit. The commercial life may be further defined by saying that it has ended in a given structure when that structure cannot be used without an annual loss due to obsolescence and age great enough to pay the prevailing interest on a sum sufficiently large to replace the structure by a new one for the same business.

This statement becomes interesting at this time to show the relation of the first cost, interest rate, maintenance and depreciation. Let

R = Income from handling goods;
I = Original investment;
r = Rate of interest on the original investment, and maintenance increments;
M = Annual maintenance and depreciation.

For the first year's operation R should equal $Ir + M$. For public property the value of R should be made such that r shall equal only the prevailing rate of interest. If 25 yr. is the assumed commercial life of the structure, there would be no advantage in building any part of it to last a longer period. The annual cost, $Ir + M$, is the first term of a progression in which 25 is the number of terms and $1 + r$ is the ratio. The serial sum of this maintenance and interest is for $r = 0.05$,

$$\frac{(Ir + M)(1.05^{25} - 1)}{(1.05 - 1)} \qquad (1)$$

Expression (1) reduces to

$$2.4\,I + 48\,M \qquad (2)$$

The total cost of a structure for the period of 25 yr. will then be (2) plus the initial cost I, or

$$3.4\,I + 48\,M \qquad (3)$$

Of two structures having equal facilities for operation, the one that will make expression (3) the smaller is the more economical type of construction to be used.

*Resident Engineer, Steamship Terminals of Connecticut Rivers, Harbors and Bridges Commission, New London, Conn.

To further illustrate assume that a wood wharf structure costs $1.50 per sq.ft.; that a steel one covered with concrete and assumed to be permanent costs $4 per sq.ft.; that depreciation and maintenance on the former is $0.10 per year, and on the latter, $0.00. By substituting in the expression (3), the economic advantage of the former is as 9.91 is to 13.60. That is, the cost of each square foot of the timber wharf 25 yr. hence is $9.91, whereas the cost of each square foot of permanent construction would be $13.60. There is an advantage of the concrete construction in insurance against fire and a point in favor of the wood construction in this case based on the assumption that there would be no maintenance for the concrete-steel construction which, of course, would not be literally true.

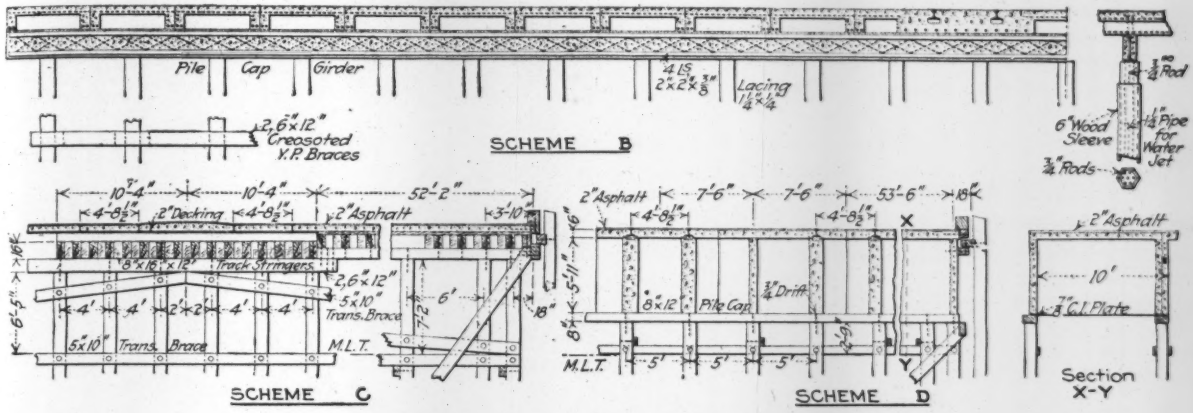


FIG. 2. ALTERNATE DESIGNS FOR NEW LONDON STATE PIER
None of them was accepted

If in (1) r is made 0.04, (2) will become $1.66\,I + 41.50\,M$, and (3) will become $2.66\,I + 41.50\,M$. That is, the cost of one square foot at the end of 25 yr. would be $8.14 for the wood construction and $10.64 for the permanent construction. The effect of the change in interest in this case from 5 to 4% illustrates the general principle that with a fixed economic advantage the lower the rate of interest the more permanent the type of construction feasible.

Since even the substructure of a pier is more or less complicated, in order to find the most economic design, it has been necessary to make preliminary designs and estimates of several different types. These were known as schemes A, B, C, D and E.

Alternative Designs; Steel-Cylinder Type

The foundation of the site of the proposed structure is sand, gravel and boulders. In some places the material is light. It required 30 to 40 ft. of penetration of a 13-in. pile to bear 20 tons as given by the *Engineering News* formula. This uncertainty of the foundation made the adoption of a complete filled pier with deep retaining walls out of the question. For the purpose of computation for the different schemes shown, a pier 125 ft. wide and 1,000 ft. long, with two bulkhead sections, has been assumed.

The subfoundation for the pier known as Scheme A consisted of steel cylinders 4.5 ft. in diameter driven as an open caisson over nine piles which are cut off at the

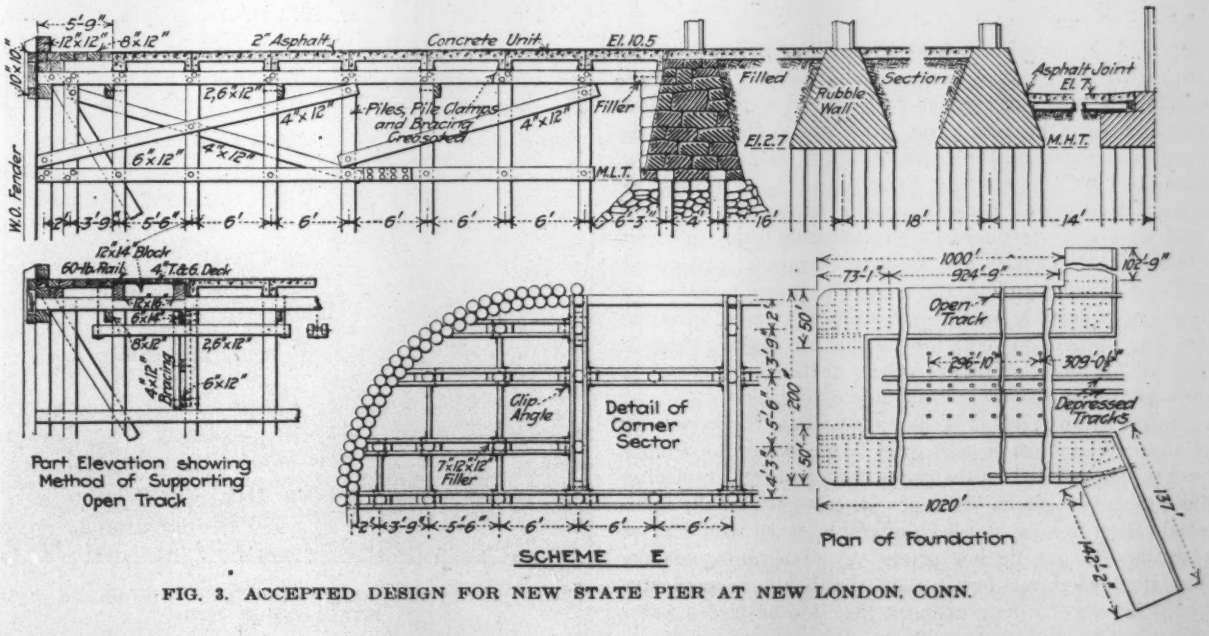


FIG. 3. ACCEPTED DESIGN FOR NEW STATE PIER AT NEW LONDON, CONN.

elevation of the harbor bottom. Concrete is then to be deposited around these piles with a drop bucket, from 2 to 12 ft., and allowed to set; after which the water is pumped out of the caisson and the remaining space filled with concrete. The top of the caisson is at elevation 3 ft. above mean low tide. A stone cap is placed on top of the concrete pier and the steel floor system placed directly on this cap. The range of tide at New London is about 3 ft. A stone cap was used in preference to concrete, as it would better resist the action of sea water during the winter months. All steel is protected with a covering of concrete.

The only questionable element in this type of construction is the concrete that is poured around the pile tops to seal the caisson. In soft soil it would be necessary to make this section sufficiently deep to balance the total hydrostatic head. This would require 15 ft. of concrete. Because of the method of pouring this concrete, the strength of it would be in doubt. In addition to this there is a possibility, if not a probability, that the boulders that exist in the soil on the proposed site would make trouble in keeping the piles in clusters close enough to permit the cylinders to pass over their tops. Since these piers are designed to retain the total load of 240 tons, it is necessary to bring this with steel girders to the pier, greatly increasing the cost of the steelwork over what it would be for spans of one-half or one-third those given. In spite of the uncertainty and extra cost of the type of construction mentioned, many engineers have used it during the last few years, doubtless believing the disadvantages less than for those of many other types that might have been used under like circumstances. For the comparison of all schemes shown, r is assumed to be 0.05. The estimated cost per square foot is $3.25 for Scheme A. This design is not shown in the accompanying sketches.

The maintenance is assumed as follows: Asphalt surface, $0.01 per sq.ft. per yr.; fender system, $0.01; concrete, etc., $0.004; total, $0.024. Inserting these quantities in (3), the cost for the period of 25 yr. is found to be $12.20.

Creosoted-Pile Substructure

The next type investigated, shown as Scheme D, contemplated the use of creosoted piles driven in bents 10 ft. apart with piles 5 ft. on centers. These were cut off at 3 ft. above mean low water and capped with a timber and a cast-iron bearing plate. Obviously, the cap mentioned and some of the piles, at least once during the period of 26 yr., would need to be replaced. The replacement of timberwork beneath a solid-concrete deck is difficult and very expensive. To overcome this difficulty a box unit of concrete was used, to be precast and cured in the yard and set on the cast-iron bearing plates by means of a derrick lighter. Pipe sleeves are inserted at the four corners to receive eye-bolts for making fast to the unit. The joints between these units are to be calked with oakum and the whole surface covered with 2 in. of asphalt. At any time it is desired to repair the woodwork of the substructure it will only be necessary to remove the asphalt and lift as many units as required, replacing the timberwork from the top. In cases where the creosoted piling is long, the 8 ft. of piling saved would be of advantage in getting low prices. A disadvantage is the fact that whatever bracing of the bents is necessary would have to be done at mean low tide or under water

FIG. 4. CONCRETE GIRDERS ON NEW LONDON PIER CAST IN PAPER SHEETING

with a consequent increased cost of maintenance work. The first cost of this pier is $1.78 per sq.ft. The maintenance is estimated as follows: Asphalt, $0.01; fender system, $0.01; track and woodwork, $0.02; total, $0.04. This gives $7.97 as the cost of the structure at the end of the 25-yr. period.

All-Concrete and Standard Timber Types

The next type of structure investigated (Scheme B) was a complete concrete pier, concrete pile bents spaced 15 ft. apart with a concrete girder cap tying the piles

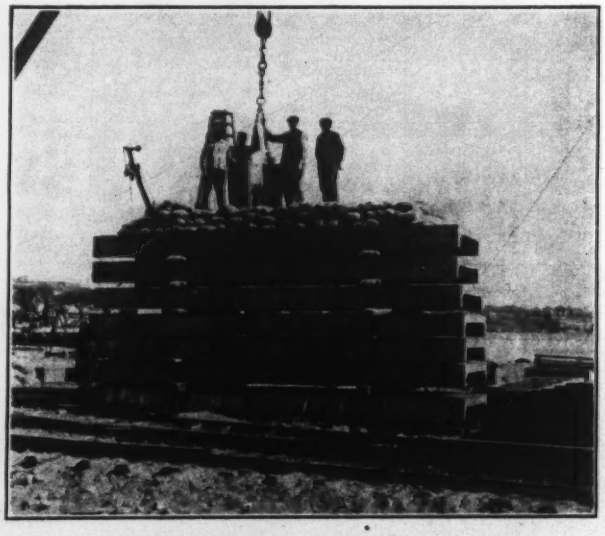


FIG. 5. TESTS OF REINFORCED-CONCRETE SLABS FOR NEW LONDON PIER

together and a concrete U-section deck, showing in Fig. 2. It was thought that one of the principal disadvantages of this type would be the action of the sea water on the piles around high tide. To overcome this action a wood sleeve is provided, lined and attached as shown. The reinforcement used in the concrete cap is structural steel tied to the concrete pile with a bolt. On the cap rest concrete unit sections for a floor system. The legs of the units are tied to the pile cap at each bearing, by means of bolts through the legs of angles and extending into the units as bond rods. The floor units are to be cast in alternate sections on a checkerboard plan to overcome the effect of expansion and contraction due to congealing of the concrete and to temperature changes. A spring-fender system of timber is placed on the outer edge of pier to absorb as much as possible of the shock due to the rolling of ships, etc.

The cost of this pier, as estimated and given, is $1.88 per sq.ft. The maintenance is as follows: Asphalt, $0.01; fender system, $0.01; tracks and concrete protection, $0.01; total, $0.03. This would give $7.65 for the cost of the structure per square foot 25 yr. hence.

Scheme C is a standard type of timber construction with a concrete deck and asphalt pavement. It involves no extraordinary feature of design. The estimated cost is $1.83 per sq.ft. for installation.

The maintenance could be rated as follows: Asphalt, $0.01; fender system, $0.01; bracing and piles, $0.013; caps and stringers, $0.03; floor plank, $0.02; total, $0.083. This would give $10.20 per sq.ft. as the cost 25 yr. hence, and it is highly probable that the maintenance charges would exceed even those given.

Type of Pier Under Construction

As a result of the foregoing computations and estimates, another type was made, shown at Scheme E, which involves some of the construction of all the other types. This design was adopted and is now building. The width of the pier is made 200 ft. instead of 125. The central core 100 ft. in width is a filled section. This fill carries two central tracks, water-piping foundations for a warehouse, etc. Increasing the width of the pier makes it possible to put a large warehouse on the central section for time storage, thereby saving to a certain extent the usual cost of transporting goods from the warehouse to the pier. The method of construction has been to first drive the piles shown as bearing piles in Fig. 3. These inclose the filled area. Riprap is then placed around these piles to the elevation of mean low tide, which is the cutoff of the piles.

The stones indicated as channel stones were placed and 2-ply "Paroid" paper pressed in between them, forming a channel directly over the top of the piles. In the channel thus formed, a reinforced-concrete girder was made containing about 1% of steel in the bottom and anchored to the piles with bond rods drifted into them (Fig. 4). These girders were put in at low tide. The paper was sufficiently strong to prevent any loss of cement. This method of forming the foundation of the wall is somewhat of an innovation, but it has worked out satisfactorily and is believed to be preferable to a wood or stone platform or grill because it insures a bearing on each pile whether it is in good alignment, cut off exactly level or not. The piles were cut off with a circular saw attached to a framework operating from the platform above.

The deck of the platform outside of the rubble wall is composed of precast sections of reinforced concrete. It will be seen from Fig. 3 that the cross-section of these units is that of the letter U. They are for the most part 22.83 ft. long, 6 ft. wide and weigh about 7 tons. They are laid to break joints, each one to rest on three supports, but not reinforced to be continuous over the middle support. Cast-iron plates are placed on top of the piles and pile clamps for the legs of the unit sections to rest on. The corrugated section of the floor thus formed allows air to circulate around the pile clamps, thereby lessening the danger of dry rot. The unit sections are not fastened to the timberwork and are laid with a slight clearance at the edges and ends, leaving joints to be filled with a mastic. This arrangement was made to add to the nonrigidity of the structure, so as to obviate, as far as practicable, damage to boats in rolling against the edge of the pier. The outer edge of the pier is of timber construction, having a solid fender system such as used in New York Harbor.

Slip Details; Progress of Work

The slips are 150 ft. wide on either side of the pier and have a depth of 33 ft. at mean low tide. A heavy cast-iron bit has been designed, having two posts tipped back from the face of the pier to prevent, as far as practicable, the hitch from slipping up and off. Fig. 4 shows the column foundations for the warehouse, also the depressed tracks on the center of the pier. The photograph, Fig. 1, shows the state of completion of the work about June 1, 1915. The rubble masonry wall is nearly completed. Some of the unit floor sections can be seen as placed in their final position.

The cost per square foot of the structure is $1.38. The maintenance (the items referring to timberwork being corrected in the ratio of the platform or timber section to the filled section) may be estimated as follows: Asphalt, $0.01; fender system, $0.0056 ($0.01 $\frac{13}{22}\frac{1}{1}$); bracing and piles, $0.0033 ($0.006 $\frac{13}{22}\frac{1}{1}$); tracks and concrete deck, $0.005; total, $0.0239.

The total area of the pier is 221,000 sq.ft. The total cost for the 25-yr. period, on the basis previously computed for the other sections, is $5.85.

Tests of the Unit Floor Slabs

The concrete used for the floor units is a 1:2:4 mix. The sand used is from Long Island; the stone, local gravel or crushed trap rock. A unit made November, 1914, with gravel aggregate, local sand and Alsen cement, was tested to destruction. The sand ranged from very fine to gravel ½ in. in diameter. The stone was crushed cobbles ranging from crusher dust to stone 2½ in. in diameter. The unit was supported on three carefully placed supports similar to the pile caps, leaving two equal spans. A load was applied at the middle of these spans. The loads were applied in separate increments by placing units as shown in Fig. 5.

1. Total load, 52,349 lb.; stress in concrete, $c = 242$ lb. per sq.in. (computed); stress in steel, $s = 10{,}340$ lb. per sq.in. (computed). There were no cracks in the unit. It was acting as a continuous beam under this load.

2. Total load, 65,925 lb.; $c = 323$, $s = 13{,}830$. There were no cracks in the unit.

3. Total load, 79,170 lb.; $c = 391$, $s = 16{,}700$. The tensile strength of the concrete over the middle support

dropped a few minutes after this load was applied. For the rest of the loads the unit acts as two simple beams.

4. Total load, 92,821 lb.; $c = 729$, $s = 31{,}100$. Small diagonal cracks appeared leading from the point of application of the loads.

5. Total load, 109,922 lb. A short time after this load was applied, the tension steel pulled through the leg, rupturing it. The unit failed by lack of bond on the rods near the supports. This perhaps would have been greater had the leg of the unit been thicker. There were two rods, 1 sq.in. each, in a 7-in. leg. The bond on the rods computed as 151 lb. at rupture. The shear through the legs computed at 78 lb. The equivalent uniform load on the unit would have been about 1,600 lb. per sq.ft.

Engineers and Contractors Engaged

This work is being carried out under the supervision of the Rivers, Harbors and Bridges Commission of the state. William T. Donnelly, of 35 Battery Place, N. Y., was employed as consulting engineer of the project. W. E. Clarke, formerly construction engineer for the United Fruit Co., located at Bocas del Toro, Panama, was employed as resident engineer and is in charge of the work.

The T. A. Scott Co., of New London, has the general contract, and the New England Construction Co., of Springfield, Mass., has the subcontract for the masonry.

The compilation of the material and studies presented here were made by G. L. Bilderbeck, Assistant Engineer, under the direction of the writer.

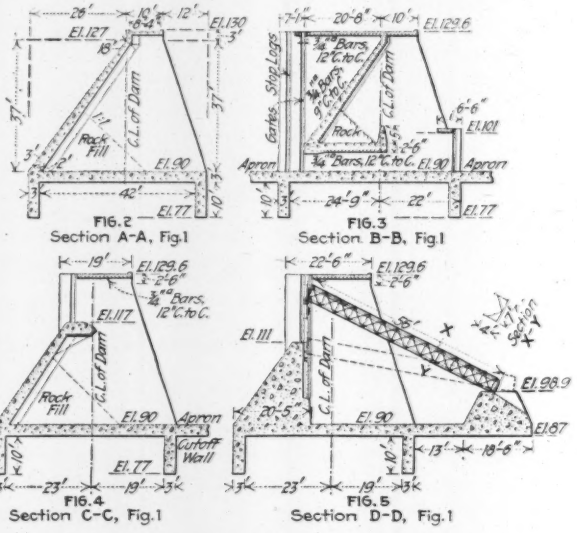
$101,000 Hollow Concrete Dam, St. Francis River

The $400,000 water-storage and regulation scheme proposed by the Quebec Streams Commission for the St. Francis River in the Province of Quebec was briefly noted in *Engineering News* of Sept. 2, 1915, p. 457. Contract for the storage dam has recently been let to Madden & Co., of Quebec, for $101,000.

The regulation scheme briefly comprises a large storage dam at the outlet of Lake St. Francis and the purchase of about 4 sq.mi. of land to be flooded and power rights now held by individuals on Lake St. Francis and Lake Aylmer (just below). By raising the water level of Lake St. Francis 27 ft. the storage capacity will be increased by 438 sq.mi.-ft., making it feasible to regulate for a constant discharge of 600 sec.-ft., which is an increase of 500 sec.-ft. over the present minimum.

The total fall along the river is about 900 ft., so that about 41,000 hp.-yr. additional is possible; but only 200 ft. available head is now utilized, so that the increase of power from the scheme will be about 6,000 hp.-yr. The present power users will be charged a sum per horsepower of increased capacity sufficient to cover the investment and maintenance of the storage scheme.

The dam itself, some typical sections of which are shown in the accompanying sketches, is to be of a hollow type, with reinforced floor slabs supporting a series of buttresses, which in turn on the upstream side carry a deck slab, flat on the outside face and arched on the inside (15-ft. radius). The buttresses also carry a concrete roadway.



FIGS. 2 TO 5. SECTIONS THROUGH THE ST. FRANCIS DAM

Fig. 2—Through typical panel. Fig. 3—Through sluices. Fig. 4—Through spillway. Fig. 5—Through log slide

The buttresses are 5 ft. thick and are on 20-ft. centers generally. In the gate section the buttress faces are vertical and have stop-log and gate grooves, as shown in Fig. 2. At the log sluice also the buttresses have a vertical face and show stop-log and log-slide grooves. The deck is carried on seats. The deck slab has the slope of the buttress faces, except for a vertical top section. At the log sluice a solid wall replaces the deck slab, as shown in the section, Fig. 5.

The floor slab varies in thickness from 15 to 36 in. The reinforcement is of ¾-in. square twisted steel bars bent so as to be 3 in. above the bottom of the slab at buttresses and 3 in. below the top of the floor between buttresses. The inner corner between the deck and floor will be filled with rock and faced with mortar. The two cutoff walls will be 3 ft. thick and 10 ft. high. At the end panels they are joined and run as a bulkhead into the banks. Cutoff walls and floor are monolithic. For floors, buttresses, aprons, etc., the concrete will be a 1:2½:5 mix, and for the deck slab and arches a 1:2:4 mix.

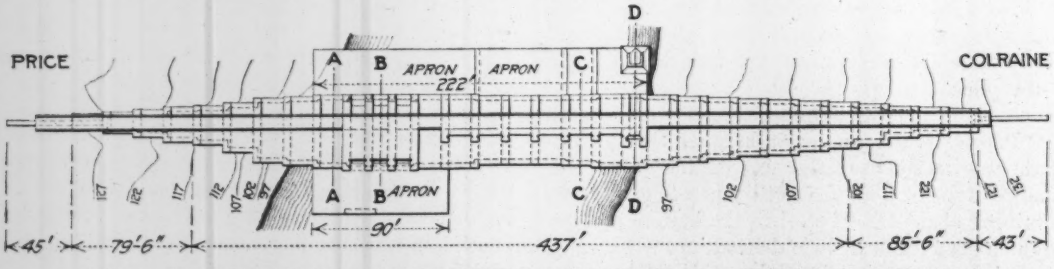


FIG. 1. PLAN OF HOLLOW CONCRETE STORAGE DAM, ST. FRANCIS RIVER, QUEBEC

The river bed at the site of the dam shows stiff clay, with small boulders and gravel. Solid rock was found at 14 to 48 ft. During construction the river will be diverted through a cut channel and the bed excavated for foundations in hardpan (2 ft. before natural bottom).

The work is being done under the direction of the Quebec Streams Commission, Hon. S. N. Parent, Chairman; O. Lefebvre, Chief Engineer, and Arthur Amos, Hydraulic Engineer.

Lining a Railway Tunnel by Compressed Air

The concrete lining of the double-track Diana tunnel on the Louisville & Nashville R.R. has been placed by the pneumatic system. The tunnel is on the new line to Birmingham, Ala. (*Engineering News*, Feb. 5, 1914). It is 1,520 ft. long, in limestone and shale, and required timbering throughout. For about 300 ft. at the north end this timbering had to be removed in advance of the concrete, but for the remainder of the distance the tunnel section was large enough for the lining. Traffic was handled on one track.

Steel forms of the Blaw type were used, with steel plates for the side walls and 2½x5½-in. pine lagging for the arch. The sides were stiffened by vertical I-beams, against which were two lines of horizontal timbers secured to the rock by anchor bolts. Struts from the I-beams carried transverse members, on which were mounted jacks supporting the arch ribs. There were two sections of the forms, each 35 ft. long, traveling on rails. The sides of the forms abutted against the footings, which were built in advance of the forms.

The mixer plant was in the approach cut, about 75 ft. from the portal, working first at the south end and then at the north end. Spoil from the tunnel was crushed to form the coarse aggregate. Sand and cement were delivered by cars and hauled up incline tracks to the top of the material bins. The materials were supplied to a measuring hopper, which delivered 10-ft. batches to the mixer chamber beneath.

An 8-in. steel pipe was laid along the subgrade from the mixer to the forms and then turned vertically, ending in a Y with two 8-in. branches. These branches led into the crown of the arch for about 20 ft., each ending in an elbow at the side wall, with a 9-ft. length of 8-in. hose.

PLACING CONCRETE LINING IN A TUNNEL UNDER RAILWAY TRAFFIC

The arch ribs are carried by jacks on the horizontal beams. The diagonal line in the rear is the concrete pipe to the arch

Men with ropes pulled the hose to and fro to secure even distribution of the concrete. The concrete was deposited on one side till the wall was 2 to 6 ft. high; then it was diverted to the other side, in order to balance the pressure on the forms.

For concreting the arch, the hose, elbows and one branch pipe were removed and a short piece of hose attached to the remaining pipe. When the concrete reached the end of the hose, this was removed and the concrete blown in directly from the pipe. The forms were left in place for sixty days at first, but afterward the time was reduced to four days.

Where the temporary timbering was removed the cavities were filled with a packing of 75% limestone screenings and 25% sand, blown in by compressed air.

The work was done in 41 sections of 35 ft., containing about 250 cu.yd. each. The concreting of a section averaged 24 hr., with a minimum of 15 hr. The distance for conveying the concrete by compressed air averaged 400 ft., with a maximum of 925 ft. The force consisted of 13 men for concreting and 10 for constructing the footings and shifting the forms.

The work was done by the Meacham Contracting Co., of Hopkinsville, Ky., using the McMichael pneumatic plant supplied by the Concrete Mixing and Placing Co., of Chicago. It was under the supervision of H. C. Williams, Chief Engineer of Construction, Louisville & Nashville R.R. George B. Bryson was Resident Engineer.

Oakland Municipal Garage

The City of Oakland, Calif., has a municipal garage, established in 1913, which has the care of all motor cars owned and operated by the city except those used by the fire and police departments. There were 18 cars when the garage was started and there are 40 cars now. Operating costs have been made the subject of considerable study and are presented in the 1915 annual report of the Commissioner of Streets.

The functions of the garage have been increased from mere daily care and minor repairs to the full duties of a repair garage, requiring a modern machine shop and an increase in the number of machinists, but resulting in a lower cost per mile in the upkeep of the machines. The garage does ordinary repairing, thorough overhauling once in each 12 or 18 months, body building, upholstering and even the making of parts and rebuilding of entire cars.

A complete check is kept on the use of the machines by an "in-and-out" register. Complete check is kept on the handling of the machines by the "efficiency marking system on auto drivers"; and the segregated cost against each machine gives a complete check on operation.

The table shows the analyzed cost data. These records are kept for the double purpose of checking the efficiency of the garage, car and driver and to serve as a basis for apportioning the cost among the various departments of the municipal government. Comparison with garage-operating costs elsewhere shows these to be unusually low.

COST OF OPERATING MOTOR CARS, OAKLAND, CALIF.

	1913-14	1914-15
Maintenance cost	$17,541.34	$20,101.78
Number of cars	35	40
Average miles per car	6,683	7,066
Average cost per car	$501.17	$502.54
Average cost per mile	.075	.071
Average miles per gallon gasoline	.....	13.78

Field and Office

Placing Monolithic Lining on Reservoir Bank

Many engineers in the Northwest have contended that the climate is so equable as not to call for expansion joints in reservoirs. L. D. Kelsey, City Engineer of Aberdeen, Wash., had the courage to construct a 7,250,000-gal. reinforced-concrete reservoir without expansion joints. The reservoir is 160 ft. square on the bottom, has slopes of 1 on 2 and a depth of about 25 ft. from the bottom to the top of the embankment. The embankment had to be made by a fill of sand and clay and was settled as the dirt was placed by keeping water from a hose running upon it. This fill was allowed to stand over winter. When the slope and the top were dressed off in the spring the fill was found to be thoroughly compacted, and careful levels showed that no settlement had occurred.

PLACING REINFORCED-CONCRETE RESERVOIR LINING, ABERDEEN, WASH.

The lining of both the bottom and slopes was 6 in. thick, with a toe at the bottom and a parapet at the top of the slope. The lining consisted of a 5-in. concrete base, using one sack of cement to 5 cu.ft. of concrete, and a 1-in. top of 1 part of cement and 2½ parts of sand. The reinforcing (see view) placed at the middle of this 6-in. slab was No. 28 triangular mesh, lapped 4 in. The joints between days' work were made by thoroughly wetting the old surface and dusting with dry cement before the new concrete was placed. The concrete was kept wet one week after placing.

No cracks have appeared except slight hair cracks at the joints between days' work, and no leakage of any noticeable amount has occurred.

Contract Plans for Sewer Work at Columbus, Ohio

The method used in presenting data on the construction plans of the new south-side storm sewer at Columbus, Ohio, appears to have many advantages. Roughly, there is a main storm sewer 6,700 ft. long, of concrete construction, varying in diameter from 108 to 120 in., with a system of connecting laterals varying in size from 12-in. pipe to 72-in. concrete, comprising an area of about 1 sq.mi.

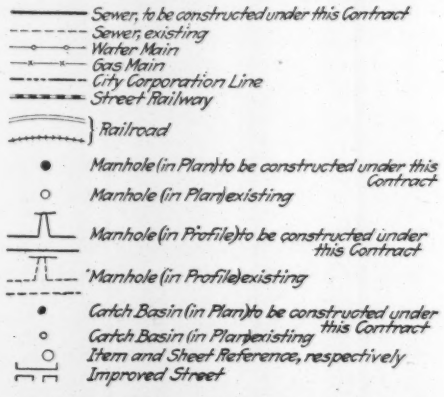

Sewer, to be constructed under this Contract
Sewer, existing
Water Main
Gas Main
City Corporation Line
Street Railway
Railroad
Manhole (in Plan) to be constructed under this Contract
Manhole (in Plan) existing
Manhole (in Profile) to be constructed under this Contract
Manhole (in Profile) existing
Catch Basin (in Plan) to be constructed under this Contract
Catch Basin (in Plan) existing
Item and Sheet Reference, respectively
Improved Street

FIG. 1. SYMBOLS USED ON STANDARD SEWER PLANS OF COLUMBUS, OHIO

There are 12 sheets of standard plans, 14x39 in. each. Blueprints of these are folded to 7½x5 in., to just fit the pocket, the title coming in the lower right-hand corner, so that by pressing back the corner of the outside fold the title of each sheet is always exposed. Sheet 1

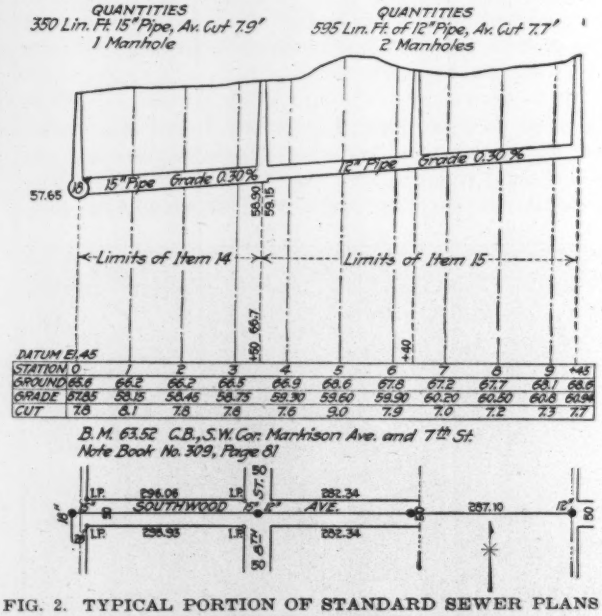

QUANTITIES
350 Lin. Ft. 15" Pipe, Av. Cut 7.9'
1 Manhole
QUANTITIES
595 Lin. Ft. of 12" Pipe, Av. Cut 7.7'
2 Manholes
15" Pipe Grade 0.30%
12" Pipe Grade 0.30%
57.65
Limits of Item 14
Limits of Item 15
DATUM EL. 45
STATION 0 1 2 3 4 5 6 7 8 9 +45
GROUND 65.6 66.2 66.2 66.5 66.9 66.6 67.8 67.2 67.7 68.1 68.5
GRADE 57.85 58.15 58.45 58.75 59.30 59.60 59.90 60.20 60.50 60.8 60.94
CUT 7.8 8.1 7.8 7.8 7.6 7.0 7.9 7.0 7.2 7.3 7.7
B.M. 63.52 C.B., S.W. Cor. Markison Ave. and 7th St.
Note Book No. 309, Page 81
SOUTHWOOD AVE.
296.06
282.34
287.10
298.93
282.34

FIG. 2. TYPICAL PORTION OF STANDARD SEWER PLANS OF THE CITY OF COLUMBUS, OHIO

is an index plan of the whole system, showing the size of the various sewers and the sheet number where the details of each section are to be found. Besides these, Sheet 1 contains a list and description of the 52 items of the contract and a list of the items on which alternate bids will be received. The symbols shown in Fig. 1 are used on this plan.

Other sheets, with the exception of Sheet 10, show plans, profiles and details of itemized sections of the different sewers. Sheet 10 shows the standard cross-sections of all sewers. Fig. 2 is a good example of the method in which detail data are presented, which seems to be a particularly handy form for the estimator. Separate bids were received on each of the 52 items and on the alternate designs for the same item.

The City Engineer of Columbus, under whose supervision the plans were prepared, is Henry Maetzel, with W. J. Weaver engineer in charge, and the contractor is the J. C. Carland Co., Toledo, Ohio.

Joints for Wrought-Iron and Steel Pipe*

By R. S. Lord†

The joints in use on wrought pipe have until recent years been threaded. Many different shapes have been used for making the couplings, with some variation in the threads cut. A few varieties will be noted.

Threaded-Joint Pipe Couplings

The standard pipe coupling, Fig. 1, has a comparatively short body with threads out to the extreme edge and is not intended for heavy pressures or difficult conditions. The line-pipe coupling, Fig. 2, is larger and heavier and, in addition to its long bearing of threads, has an overhanging extension, which is intended as a convenience for calking in case of leakage, as well as a reinforcing section. This feature is emphasized in the air-line coupling, Fig. 3, in which the recess for lead is large enough to permit thorough calking. This leading feature is used because of the fact that it is impossible in diameters 4 in. and larger to make screw joints that will hold high pressure of air and gas without probability of leakage.

The inserted joint, Fig. 4, is used in casing where a common collar would obstruct the passage within a larger casing. The flush joint, Fig. 5, is used in the same way where external smoothness is more important than strength. Neither shape is used in pipe lines.

Flanges are rarely used on wrought pipe, except in steam piping and for connections to valves and fittings. In large diameters it is difficult to make the thread in a flange tight; peening the end of the pipe for the purpose of stopping leakage is a common practice. The modern Van Stone joint is an improvement in flanges. It is made by belling out the end of the pipe at right angles to the axis of the pipe. The flanges, without thread but machined to the curve of the bell, are of course put on the pipe before the end is expanded. The gaskets are inserted between the ends of the pipe rather than between the cast flanges.

In the early days of the natural-gas business, engineers, realizing the weakness of large screw couplings, turned to lead for sealing joints. The Kimberley joint, Fig. 6, was a heavy forged collar with ample lead space for calking, but without provision for locking. The Converse joint, Fig. 7, was made by locking wrought pipes in a cast-iron hub by means of projecting rivets on the pipe made to engage in slots inside the hub. When locked the joints were leaded. This joint was in common use for large pipe lines 25 years ago; most of them now have rubber clamped against the lead to stop leakage, or the lines have been relaid with a rubber-sealed joint. The McKee coupling was somewhat like the Converse; the lock was made by a series of rivets set in spiral lines, inserted in spiral grooves, and leaded.

The Matheson joint, Fig. 8, is of bell-and-spigot design, the bell being formed by expanding one end of the pipe sufficiently to inclose the spigot end and the necessary lead space. This joint is in common use for water lines in the West, where freight rates run up the cost of cast iron. When used for gas lines, rubber is usually clamped on to insure tightness as well as rigidity.

Rubber-Packed Pipe Couplings

Rubber-packed couplings have opened a great field for inventors. While many differences in detail are to be noted in the different couplings, they seem to divide readily into two general classes—those intended to be packed by the pressure of fluid carried and those packed by force applied externally, working against the pressure within.

From 1891 to 1905 all Dresser couplings (one of the earliest) were made with cast-iron sleeves, or center rings, as they are commonly called. The first followers, or end rings, were of cast iron, but the change to malleable iron was made quite early. Where cast-iron center rings overlapped the rubbers and the end rings merely served to crowd the rubbers forward into the recesses under the center rings, this proved to be too much of a strain on the latter.

The Dayton coupling, Fig. 10, changed the faces of the center rings and rubbers so that they met on a slope, tending to drive the rubber down against the pipe. The Hammon coupler, Fig. 11, placed the rubber under the projecting edge of the center and the compression was applied by crowding the rubbers forward into the stuffing-box, thus putting the heaviest strain of the compressed rubber on the center ring. The most radical change in the early Hammon coupling was the eccentric feature, which made it fit equally well whether the pipes were laid in a straight line or at an angle up to 10° or 12°. The eccentric features of this coupling required an excess of material and increased the cost, but it still has many friends who, on account of its convenience, will use no other.

In the Custer type (1905) the center ring was made of steel, and the change of material was soon made by other manufacturers. The Custer coupling, Fig. 12, shows a center ring with outwardly flaring bell-shaped rubber recesses and an enlarged interior, giving great room to swing at a sharp angle, if properly controlled, so that an equal pressure can be maintained on the rubbers. In order to maintain this pressure on the rubber, it is necessary to have an end ring with broad face to close the opening, without much clearance. Rubber has

*From a paper before the Engineers' Society of Western Pennsylvania.

†Vice-President, Hope Engineering and Supply Co., Mt. Vernon, Ohio.

a habit of escaping from pressure by crawling through an opening, and the original Custer couplings were weak along this line.

From the beginning of the steel-center-ring business, the dividing stops or lugs have differed. Neither Custer nor Dayton made a square stop by which the rings could be squared up against the ends of the pipe, and the tendency to slip a lug past the edge of the pipe often made it difficult to shape up the joint as it should be.

Dresser adopted a ridge running entirely around the interior of the center ring, making a square stop that lined up the rings very satisfactorily. This method is now used by Dayton. Hammon has always used square detachable lugs forming a perfect stop to line up the pipe; they can be removed by hammering if it is desirable to use the ring as a sleeve.

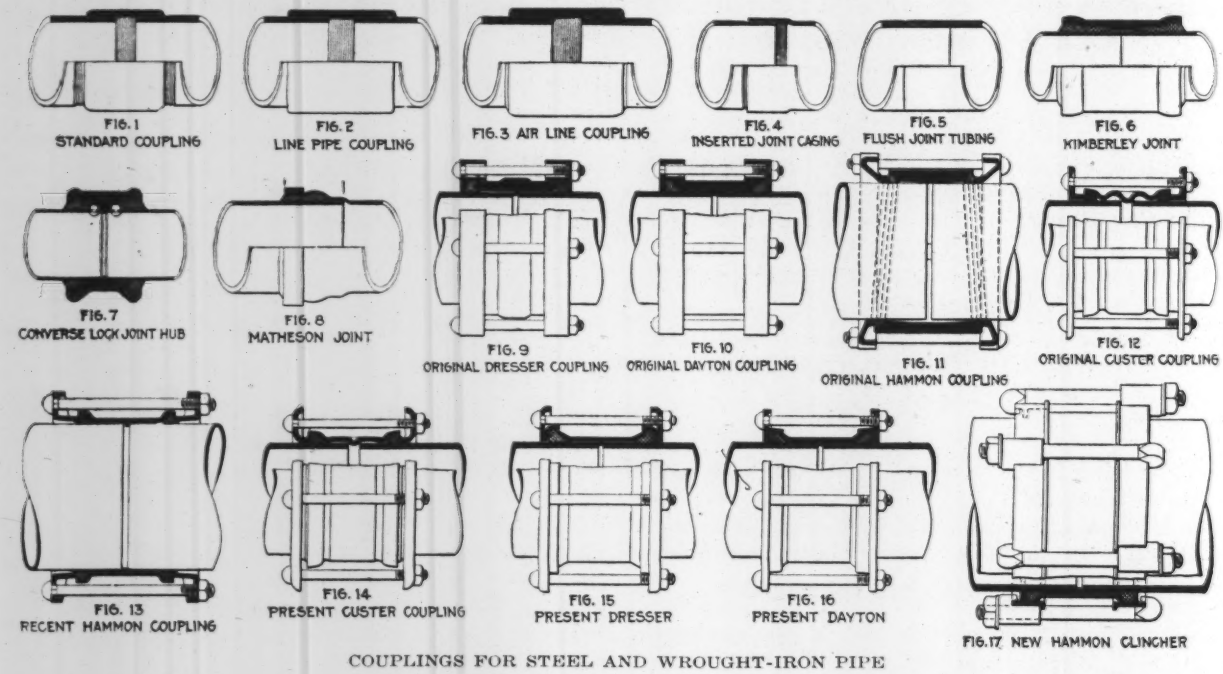


FIG. 1 STANDARD COUPLING
FIG. 2 LINE PIPE COUPLING
FIG. 3 AIR LINE COUPLING
FIG. 4 INSERTED JOINT CASING
FIG. 5 FLUSH JOINT TUBING
FIG. 6 KIMBERLEY JOINT
FIG. 7 CONVERSE LOCK JOINT HUB
FIG. 8 MATHESON JOINT
FIG. 9 ORIGINAL DRESSER COUPLING
FIG. 10 ORIGINAL DAYTON COUPLING
FIG. 11 ORIGINAL HAMMON COUPLING
FIG. 12 ORIGINAL CUSTER COUPLING
FIG. 13 RECENT HAMMON COUPLING
FIG. 14 PRESENT CUSTER COUPLING
FIG. 15 PRESENT DRESSER
FIG. 16 PRESENT DAYTON
FIG. 17 NEW HAMMON CLINCHER

COUPLINGS FOR STEEL AND WROUGHT-IRON PIPE

Improvements in Rubber-Packed Couplings

In the evolution of steel end rings very different designs have been used. It has been found difficult to get sufficient stiffness in a forged ring to resist the tendency to lean forward unduly under the heavy pull of the bolts. It is impossible to rib up and strengthen a forging, as is done in a malleable casting; consequently, when the center ring incloses the gasket, preventing any bursting strain from coming on the end ring, malleable iron is a better material for a stiff gland than steel. For this reason Hammon has manifested a preference for malleable end rings, Fig. 13, wide on the face against the gasket and sufficiently rigid to resist the bolting strain.

Custer has cut steel rings out of plate, Fig. 14, pressing them into cup shape. Dresser uses steel plate, but presses it so that the gaskets can be inclosed and a ring obtained that stands up well against the bolts. Dayton uses a special rolled shape resembling a T-bar, Fig. 16, which also incloses the gaskets. The standards of the various manufacturers as made at the beginning of this year will pack an ordinary pipe line very satisfactorily.

An entirely new coupling is being made by the Hammon company and is largely the invention of H. D. Hildebrand. The new design, Fig. 17, is made close down to the pipe and the bolts have a hook at the head end and a clip at the nut end. The pull is as direct against the gasket as it is possible to get it. The strength of the ring is made in the direction of this pull of the bolts so that it cannot be bent between the bolts. The shallow depth in radial lines will permit of better adjustment to irregular pipe than can be obtained where the usual wide flange is used. In addition there is the saving of more than half the labor of bolting up, since these bolts hook on and are tightened immediately.

The Advantages of Plain-End Pipe

The plain-end pipe is the easiest line to join together and has the tightest joint known. The pipes themselves are simple, are not likely to be damaged seriously in handling and are easily cut into short lengths if that becomes necessary. The unbroken film of mill scale over them helps against corrosion, making a thin plain-end line as durable as a heavy screwed line with its deep-cut threads and tong scars. With a good coating of preservative paint, it is possible to insure long life with much lighter than standard pipe.

The joints, when properly constructed and anchored, are capable of making bends up to 5° without danger of leakage. This absolutely tight condition insured by a good gasket is perhaps the strongest point in favor of such a line, but the fact that it can give and take under change of temperature or settlement without making a break puts it in a class by itself.

As to strength, repeated tests of various modern couplings have shown them to be in all cases stronger than the pipe itself, although made of the same thickness of material. In the one point of rigidity, it must depend upon some method of anchorage, such as a covering of earth in the ditch or, if exposed on a bridge, a secure

fastening to the bridge itself. It would be unwise to recommend plain-end pipe for exposed lines carrying heavy pressures without anchorage, as slight deflections quickly upset the balance of pressures and the line will most surely be forced apart.

The most vital point in a plain-end coupling is of course the gasket. Under conditions of air, water or dry gas carried in a line, the rubber will doubtless last as long as the pipe itself and no better packing can be devised. In recent years gasoline has come to be a common thing in natural-gas lines. Common rubber cannot long resist gasoline, but fortunately rubber compounders have come to the rescue with new compounds.

While natural gas has been the particular field of plain-end pipe, it has been used to some extent on artificial gas, air, water and oil lines, and the demand in these fields seems likely to increase. It is a most convenient line for carrying air for quarries and mines, the saving in power through the elimination of leakage making it especially valuable. If the improved gaskets stand as well against drip oil as they do against gasoline, they will open a wide door to the artificial-gas business. Some manufacturers are making such claims for their material, and the case is likely to be proven soon.

Within the past year the City of Pittsburgh has laid several plain-end water lines on bridges, among them being two 16-in. mains across the new Bloomfield Bridge and two 24-in. across the new Point Bridge, the latter bridge being the first place where the bolts were clipped over the end rings. The success of these lines leads to the hope that the demand for such water lines will multiply rapidly.

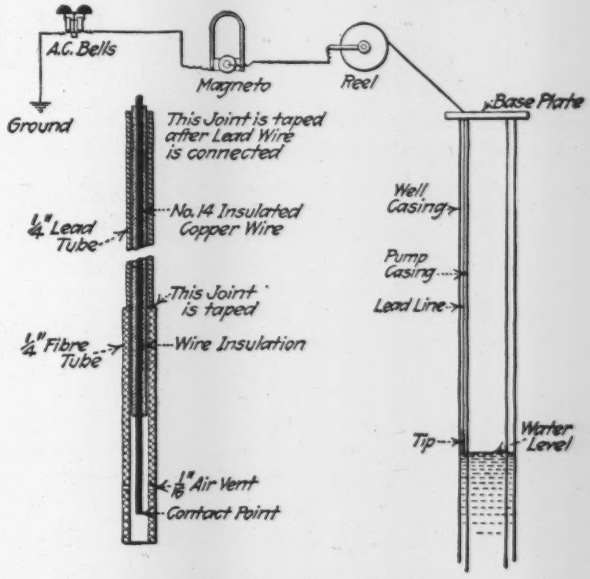
Measuring Well-Water Levels Under Difficulties

By H. W. Keith*

The writer was at one time confronted with the problem of ascertaining the draw-down and refill curves of a deep well of small diameter operated by a plunger type of pump. The well, being slightly out of plumb at several points, had clearances between the well and pump casings at these points too small to allow the use of the "air-line" method. To overcome this difficulty, a portable apparatus which requires only ⅜-in. clearance or less was devised by the writer. It comprises an electric circuit, which is completed when the water level is reached, with a hand magneto and a bell.

The tip, as shown in the sketch, is a piece of No. 14 insulated copper wire, inclosed on one end by a lead tube for weight and on the other end by a fiber tube, the insulation here being stripped back about 1½ in. to form a contact point. An air vent is drilled out about 1½ in. above the contact point. The tip is given a good coat of shellac, after the joint between the lead and fiber tubes has been firmly taped. The end of the tip, as shown in the sketch, is connected to the leading wire, a small rubber-covered flexible cable, the length of which is governed by the depth of the water plane, and this is graduated in 5-ft. intervals.

A case, about 9x9x18 in. and divided into two equal compartments, contains the apparatus. In an upper compartment are placed a set of alternating-current bells and a hand magneto; in the lower part is the reel for the lead wire. In making the reel a steel axle is used, one end of the sounding cable being soldered to it and a constant connection obtained through the bearing of the axle. This bearing, the magneto and the bells are connected in series, as shown in the sketch. The other side of the bells is connected to a metal plate which forms the bottom of the case and is used to obtain a ground



ELECTRIC WELL-SOUNDING OUTFIT

connection. In operation, the case is rested on a good ground connection; the tip, with lead wire, is dropped down the well and the magneto operated. When the tip reaches the water level, the circuit is completed and the bell rings.

The writer has tested a number of pumping plants, using this apparatus, and has found that not only could he follow the water level very accurately, but that in many cases it was the only medium by which the water level could be ascertained.

Fire Damage to Steel Viaduct and Method of Repair

Judgment based on inspection was the criterion used to determine where steel had been injured by heating in the lumber-yard fire which on May 25, 1914, wrecked the Nickel Plate viaduct across the Cuyahoga River in Cleveland (illustrated in *Engineering News* of June 4, 1914, p. 1269). The main girders on a length of 450 ft. of the viaduct had acquired a permanent sag from the fire. The spans were 60 ft. and 30 ft. In the 60-ft. spans, when the sag exceeded ½ in. below horizontal (or about 1 in. total, the girders originally having ⅝-in. camber) the girder was considered sufficiently damaged to be taken out. These girders all had their webs somewhat buckled, but none badly enough to be dangerous. The 30-ft. girders, on the other hand, were condemned mainly for buckling of web.

The girders within the length affected by fire showed extensive buckling of plates, local distortion of flanges, a slight degree of sagging, and in the case of one 30-ft. tower span complete lateral buckling which stopped just

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short of collapse. The spans had crowded together when expanded by the heat and forced the viaduct out of line, and subsequently on cooling they did not return to place, but drew apart at the expansion joint. That there was a considerable amount of endwise compression of the girders is shown by the fact that all of them were shortened

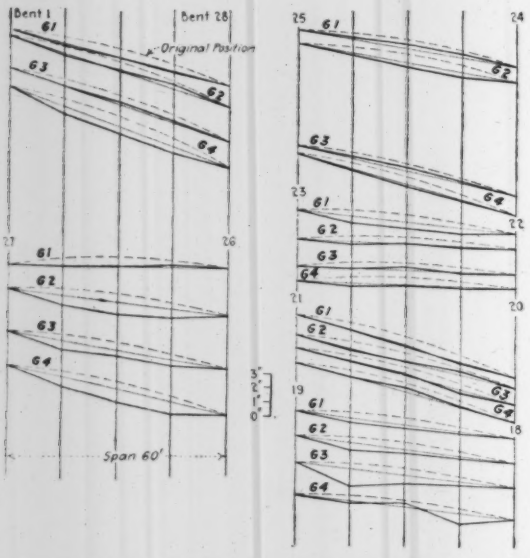


FIG. 1. PROFILES OF 60-FT. GIRDERS, NICKEL PLATE VIADUCT, SHOWING DEFLECTIONS CAUSED BY FIRE

slightly, in so far as check measurements could be made. The 30-ft. girders were shortened more in proportion to their length than the 60-ft. girders, and this resulted in drawing the tops of the towers together. The 60-ft. girders were shortened on the average ⅛ in. to ⅝ in. The track forms a 3° curve on the viaduct, and the structure was pushed 8 to 9 in. out of line at the middle of the curve.

The amount of damage from web buckling was determined by judgment on inspection. Flange damage was estimated by the sag or deflection of the girders, as already mentioned. The diagram Fig. 1 gives a plot of the measured deflections of the 60-ft. spans, the dotted lines indicating the original cambered position of the girders. The sag below horizontal averages ¼ to ½ in., but in one case amounted to 1⅛ in., or a total deflection of over 1½ in. It was decided on the showing of this plot to retain all the 60-ft. girders except the one which had maximum deflection and to replace most of the 30-ft. spans.

The drawing Fig. 4 shows the replacements. The same drawing shows by a detail sketch the replacement work in one of the towers, this being selected as representative of the work done on five towers.

Pier and Tower Repair

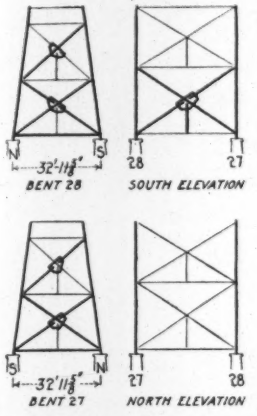
In connection with the tower work the renewal of pedestal caps was an important operation. The sandstone caps had generally been destroyed by the heat. They were replaced by precast concrete cap blocks slipped under the tower legs after jacking up the leg, or, where the column was renewed, setting the cap after removing the old column and just before setting the new.

Where the columns were damaged, but were not removed, the effective column length was reduced by putting in struts at middle of the story height, as can be seen in the sketch in Fig. 4.

There was great difficulty in pulling the distortion out of the towers. It was attempted to square up the towers after the new columns were in and riveted up, but

FIG. 2. PART OF WRECKED NICKEL PLATE VIADUCT RESTORED ON TIMBER BENTS

not always with success, and some diagonals had to be cut and shortened. The main purpose of the squaring up was to get the viaduct back into line. It was necessary to leave the column footings off the center marks both longitudinally and transversely in order to get the tops of the towers to approximately correct position. In the whole fire-stricken part of the viaduct very few loose rivets were found, which differed from expectations. A curious phenomenon, however, was that many of the rivets through girders and shoe plates were sheared off. This defect could be discovered only by close inspection, as superficially the rivets appeared to be sound.



Typical Tower Replacements.

The successful and efficient handling of the reconstruction work was due to the close coöperation of E. E. Hart, Chief Engineer, and George H. Tinker, Bridge Engineer, of the railway company, and the officials of the King Bridge Co., who carried out the work. The construction forces, both in clearing the wreckage and opening traffic and in the subsequent reconstruction, worked solely with a view to the urgent railway requirements.

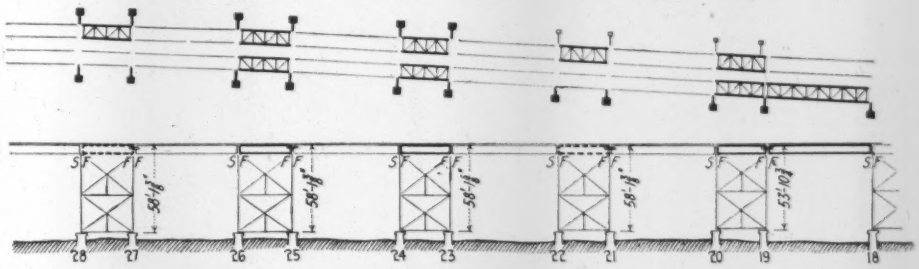


FIG. 4. NICKEL PLATE VIADUCT REPLACEMENTS AFTER LUMBER-YARD FIRE

There was an unusual amount of steel cutting required for clearing the wreckage of the city viaduct that had fallen over the tracks; 5,000 cu.ft. of acetylene and 15,000 cu.ft. of oxygen were used. The price paid the contractor for this clearing was $9.50 per ton loaded on cars, railroad weight.

Just after the fire an unusual factor was the caustic nature of the water in pools on the ground. Enough lye had been dissolved from the ashes to make the water uncomfortable and damaging in contact with skin. The wreckers and erectors soon learned to avoid these pools.

FIG. 3. ONE OF THE UNDERPINNING OPERATIONS IN RESTORING NICKEL PLATE VIADUCT
Blocking on tower leg, ready for jacking-timbers. New precast concrete block pedestal cap is at left of base

NOTES

Handling Bulk Cement On A Road Contract—On a concrete-road job about 10 mi. long in Lake County, Ohio, the contractor, the Williams & Little Co., of Cleveland, Ohio, is using cement in bulk, and A. E. Williams of the firm has devised some special methods of handling the cement from car to mixer. When a car of bulk cement is opened on the siding, it is partly emptied by hand, using lime scoops to pour the cement down chutes to cement-tight wagon boxes, which are so placed that the top is at the car-floor level. When a narrow lane has thus been opened in the car from door to door, an unloading buggy is introduced and is filled by forcing its lip into the cement face as far as possible and topping off by using a specially designed hoe. This buggy then completes the filling of the wagon boxes. The wagons are then driven to portable cement-storage boxes, which are located at intervals along the road ahead of the mixer and are moved ahead from time to time as the concreting work passes them. The cement is unloaded into these boxes by use of lime scoops and chutes in a manner similar to the car-unloading process. The boxes are so designed that unloading can be done with equal rapidity whether full or nearly empty. From the portable boxes the cement is handled by scoops into barrows graded to batch proportions and the barrows are wheeled to the mixer.

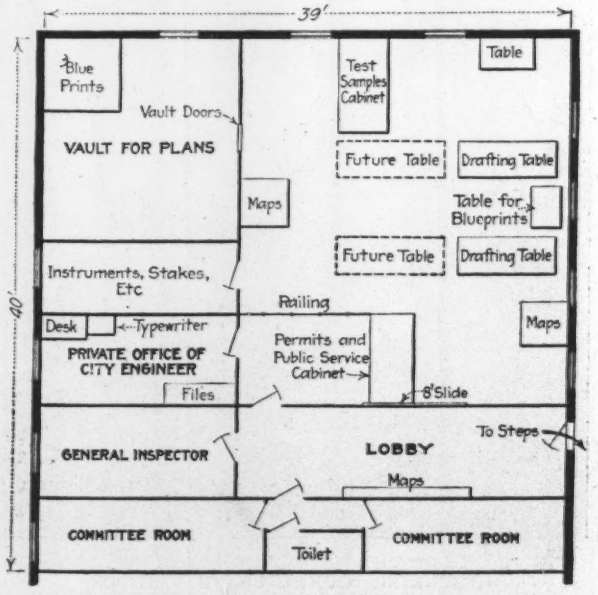
Diagrams for Determining Pavement Crowns—For using the same formula to determine the crown of pavements (where the curbs are at different elevations) given by Charles W. Barber in "Engineering News," of Sept. 9, 1915, page 509, William A. Lyon, of the City Engineer's Office at Syracuse, N. Y., uses a chart, which eliminates computations. The practice there is to make the pavement cross-section parabolic, with the rate of transverse grade d independent of the longitudinal grade. The crown is made equal to $^1/_{60}$ of the street width on a street with opposite curbs level, and where the street is laterally inclined the crown is taken as $^1/_{60}$ of the width of a theoretical street, the half width of which is the horizontal distance from the lower gutter to the crown point. Expressed algebraically, $C = \frac{W + 2X}{60}$. Two formulas have been used for computing the crown shift X. The first, $X = \frac{15\,Wh}{2\,W - 30\,h}$, has recently been discarded in favor of $X = 15\,h$. The latter formula gives a slightly greater shift under ordinary conditions than the first.

SHOW-WINDOWS IN THIS SIDEWALK SHED KEEP BUSINESS ALIVE

Sidewalk Shed with Show Windows—The builder's sidewalk shed illustrated herewith is probably the most elaborate one ever constructed. It was built recently in front of the New England Building, Cleveland, Ohio, the first story of which is occupied by a large book store. The building is being changed into an arcade, the book store to occupy one side, and it was important that during the rebuilding the book business should be disturbed as little as possible. The shed is developed in arcade outline, with some pretence to aesthetic effect. It stands on the outer half of the sidewalk. The inner wall of the shed is formed into show windows or display cases facing the passageway, taking the place of the store-front show windows, which are put out of service in the reconstruction. As the passage is rather dark, the illuminated show windows afford effective display. The space between the inner wall of the shed and the building front is used for the contractor's purposes, except that two inclosed walks go through it, leading from the sidewalk shed into the store.

Roof Planking Jointed with Gasoline-Driven Saw—The rig described herewith is being used for sawing the boards or sheathing on a large circular roof over an oil-storage reservoir. On account of the rafters all converging toward the center of the roof, it was necessary to nail the roofing in place with the ends of the boards overlapping and then saw through both boards to make the joint come on top of the rafters. Power is furnished by a 4-hp. gas engine belt-connected to a 10-in. circular saw and mounted on a two-wheel truck having Ford automobile wheels fitted with ball bearings and rubber tires to help take care of the vibration of the engine and to make the rig easy to move about over the roof. The engine, a 4-hp. throttle-governed high-speed gas engine built by the Cushman Motor Works, of Lincoln, Neb., has given satisfaction because of its light weight and the ease with which it is operated. A similar rig operated by electric power was described in the "Proceedings" of the American Society of Civil Engineers for August, 1915, page 1340. The gas-engine rig is superior to that one, because electric power is not always available and there are no electric wires to drag around. The machine is operated by two men and weighs complete but 240 lb. This includes the water-cooling tank. Its cost fitted to operate was $192.—E. D. Cole, Supt. of Constr.

An Uptodate City-Engineering Office is included in the interior arrangement of the new $125,000 city hall just completed in Muscatine, Iowa. Before the final plans of the building were approved City Engineer C. H. Young inspected the layout of engineering departments of a number of cities with a population of from 15,000 to 100,000. From the data thus gained the layout shown in the accompanying plan was made and accepted. The department is on the main floor of the city hall. Mr. Young was seen in his new office on Nov. 3, in the midst of moving, and expressed satisfaction over the new quarters.



CITY-ENGINEERING DEPARTMENT AT MUSCATINE

Editorials

The Culebra Slides at Panama

A masterpiece in technical reporting of facts is Governor Goethals' account of the Panama Canal slide situation. His statement carries clear conviction that the long chain of earth movements which mark the history of the canal has been handled in a capable, deliberately judging manner. Moreover, it inspires a fair degree of confidence in an early ending of serious slide trouble for the canal. The present two immense slips at Culebra, in the light of Governor Goethals' review, represent part of the final adjustment of the ground to its new slope and water conditions. Just as the thorough job of excavation done to clean up the Cucaracha slide of two years ago produced a result that upon a year's test of the permanent water conditions appears to be stable, so the removal of the present Culebra slides promises to bring quiet in this region also.

But in the matter of such earth movements as these, any definite prediction is little else than barefaced guesswork. Clays of landslip proclivities not only defy calculation, but are free from all regularity or habit of behavior by which guessing could be guided. Their performance inclines toward the unexpected. Brief reference to the nature of landslips will show convincingly that they form poor subjects for prediction and prophecy.

The classification of Panama slips given by Governor Goethals in the present report furnishes a good basis for such reference. His classification does not fall very far short of covering landslips in general. It is by no means the least valuable feature of the report.

First and simplest are slips of the kind discussed in the ordinary retaining-wall theory. These, however complex they may become, both as to physical facts and as to theory, are always of small size.

Second are the slides of the ship-launching type, where a natural slip-plane exists, on which the superincumbent mass begins to slide, under critical conditions of lubrication. Such slides are not exactly rare. They are slow-moving, and may be very large and exert enormous downhill pressure on an obstacle. Governor Goethals says that the great Cucaracha slide was of this type.

Third are slips of the plastic-flow kind, a rare and highly unmanageable type. They are represented in the remarkable series of slips in valleys of the St. Lawrence River tributaries in Quebec—most notable of them the last, the slip which overwhelmed the village of Notre Dame de Salette in the Lièvre valley in 1908. They are also represented in the recent subsidence of a large cement plant at Hudson, N. Y., as well as in the tipping of the Transcona grain elevator two years ago. According to the description of the Culebra slides, these too are of the plastic-flow type.

In these latter slips a clay-like subsoil becomes suddenly plastic—not to say semiliquid—under certain conditions of moisture and pressure. The Quebec slides, as reviewed in *Engineering News* of May 27, 1909, show the characteristic phenomena most clearly. In a typical instance, a flat river terrace, up to 60 ft. high above water, developed surface cracks, and soon afterward a large section of the bank subsided suddenly, the material below bursting out from the bank while the upper strata in part dropped nearly vertically. The Quebec slides include one or two bottle-shaped slides, where the failure area was very narrow at the river bank, while back of the bank it spread laterally like a dock basin, and the material squeezed out by the subsidence flowed out through the narrow bottleneck break of the bank.

Practically nothing is known as to the particular soil qualities responsible for the plastic-flow slips. We do not even know that notorious landslip soils differ in any way from normal clays or silt soils. We do not know whether sand mixtures play a part. The necessary careful laboratory attack on the problem is still to come.

But that high moisture content plays a determining part is well recognized. All but one of the Quebec landslips occurred in the months of March to May, and the one exception came in October after several days of rain.

Similarly, we know well that pressure is a vital factor. In the Transcona elevator case, for instance, how clear is the picture of sudden breakdown of the subsoil, crushed under its normal load! The same soil carried the same load for a long time, with no yielding; but when some slight change in its nature—no doubt due to an increase of water content—occurred, quite suddenly the bed of clay failed, flowed away laterally against the counter pressure of the surrounding soil and destroyed the support of the building. Only because we call this a foundation failure is it any different from the landslips.

The compressive loads which caused the flows seem to have been such as occur under very moderate depths of overburden—sometimes as low as 30 ft., which would hardly exceed 1½ tons per sq.ft. or such as may be caused by a not unusual surface load, as the weight of the Transcona elevator, or of the stone storage pile of the Knickerbocker cement plant.

Considering the time just prior to any one of these slides, it is obvious that not only was there no warning, but every assurance could be drawn from long-continued permanence of conditions that the soil was stable. In the Quebec valleys, for instance, farms and villages existed many years on and near the ground which ultimately subsided and flowed out. Any prediction, certainly, would be for continued stability and quiet. The slip of course disproved it, and just as surely would make any future prediction in the same region valueless. As said earlier, predictions are guesswork.

At Culebra, while the plastic-flow character is evident from the description—and Governor Goethals himself makes a clear distinction between the Culebra or "break" type and the other slips at Panama—the situation is somewhat different, and by no means equally hopeless, from the predicting viewpoint. The flows that occurred there were relatively slow movements. The soils that crushed out under the superincumbent weight did not fail

as suddenly or completely; they have no such narrow critical range as exhibited in other slides of this type. Adjustment should therefore be easier, and the permanence of the result should be more dependable.

The toe abutment of the slope is no more being cut away by canal-prism excavation. Water being already in the canal, there will be no further rise of ground-water. Moreover, the movement of the sliding ground relieved the soil stresses in large part, and with continuance of flow during excavation of the slide toe a full opportunity is given for complete relief. Adjustment to the present saturation level may therefore be expected to be complete. Subsequent reduction of load by surface excavation, as a protective measure, is likely then to have greater effect than did the prior steam-shovel terracing in the same region, which consumed millions of dollars.

Removal of the slide, says Governor Goethals' report, is going on at the rate of 1,000,000 yd. per month. The total earth to be excavated—only a small part of which is in the canal—is 10,000,000 yd. If the worst comes to the worst, if all the material slips into the prism, the canal will be closed a long time. The indications are in any event that the closure will continue a number of months.

The citizen as stockholder in the canal enterprise should accept this fact without complaint. Indeed, since the adjustment slip occurred so soon after the opening of the canal, he may feel fortunate in having the slip occur at this time, and under circumstances where the best method of removal can be applied systematically and thoroughly.

Co-operation or Amalgamation of Local Engineering Societies

"Do local chapters of national engineering societies tend to disintegrate the older type of local societies without commensurate benefit to the members of the local chapters?" This question was put in a brief editorial note in *Engineering News* of Oct. 14, 1915. Primarily to help solve this problem of possible conflict, the Cleveland Engineering Society devoted its November meeting to an open discussion of the relations between the different local associations of engineers and an inquiry as to whether coöperation, affiliation or amalgamation is what the situation calls for. It was the first step toward the formation of an intersociety committee or council, which will attempt to solve the problem.

It seemed to be the unanimous opinion of this meeting that collective effort—coöperation—was desirable, at least so far as concerned local affairs. The main question, then, is, How is coöperation to be brought about? The matter of whether local chapters of the national engineering societies are necessary when there is a strong local engineering society already in the field was apparently not considered. There is a rapidly growing tendency to establish local chapters of the national societies, and no one disputed their real need. There is evidently a useful field for all.

Should these local chapters merely coöperate through representation in a general council, should they also affiliate to the extent of using the same quarters, library, etc., or should they amalgamate and be merely sections or divisions of the local engineering society? If all the members of the local chapters could be induced to join the local engineering society, amalgamation would undoubtedly be the best solution. The recent experience of the Engineers Club of Philadelphia in increasing its membership fourfold shows what may be done in this direction by active campaigning.

The only arguments put forward at the Cleveland meeting for maintaining the local identity of the national engineering societies were that the papers and discussions would be of restricted interest and that certain discussions would be devoted to the affairs of the parent societies, and should of course be limited to members of those societies. To these the local society, with subdivisions or sections, presents no obstacles.

The real obstacles to the formation of a single united local engineering society are natural indifference, narrowness, clannishness, and perhaps the parsimony of individuals. It is generally a question of whether the local member of the national engineering society feels that he will derive enough additional benefit to pay him for joining the local engineering society. To belong to a local chapter of the parent society usually costs little or nothing additional. It is plain that a local society which will include all the local members of the national engineering societies is a sheer impossibility.

Should the established local engineering society, with its clubrooms and library, invite the affiliation of the local chapters to the extent of sharing its quarters and facilities with the members of the local chapters? If this were done, these chapter members would enjoy practically all those privileges at cut rates. Obviously that is unjust to the regular members of the local society and would remove all but philanthropic motives for active membership. Frankly, would the local society gain enough in influence and prestige to pay for carrying all these deadheads?

If it is considered necessary to preserve the individuality of the national society chapters, but one logical course is left. That is for local chapters and the local society to coöperate on an equitable basis, just as do the national engineering societies (with the well-known exception of the American Society of Civil Engineers) in New York City. They could erect and share a building or meeting rooms, could have a common library, a general council, and if desirable have general meetings for discussion and action on local affairs—at the same time preserving their organizations as individual societies and paying the costs in proportion to the use they make of the common plant and equipment.

At first glance this looks like the extinction of the local engineering society, but it would not necessarily follow. If the local society can maintain a live organization, if it can offer its members something that the local chapters do not, if its subjects of discussion are of broad and general interest—in short, if it is a real factor in both engineering and community affairs—the local engineering society will not only survive but flourish. It will never include *all* the members of the local chapters, but it will include the livest and most influential members in those chapters and a host of other engineers who do not belong to any of the national societies.

Mere members are not all that is desirable. A small but active, interested and energetic membership generally accomplishes more than a large membership lumbered up with the deadwood of the many who are indifferent and inactive. Circumstances induce and perhaps neces-

sitate this condition of affairs to a greater or less extent in the national engineering societies, but it is a condition which can and should be guarded against in the local engineering society.

It must be admitted that the really live local society is at present the exception rather than the rule, yet it is not improbable that the competition or conflict between the local society and the local chapters of the national societies may actually end in developing the local society, compelling it to broaden its activities or to succumb.

If the local society lives up to its opportunity, it will become the leader in the broad field of public usefulness and be a power in forming the public policy of its community in engineering and technical affairs. In this field the local chapters of the national engineering societies, both individually and collectively, are handicapped rather than aided by the ties which hold them to the national bodies. In this way the local society will attract to its membership, without concessions, the most desirable home members of the national societies, and it will become of indispensable assistance to all broad-minded, public-spirited engineers, whom we all hope to see the men of affairs of the future.

Eccentric-Loading Questions

A tendency toward iconoclasm is a good sign. It means healthily active thought, especially when it concerns the structural field, where strict rule of calculation usually governs. In this light it is a welcome fact that a bit of radical dissent is voiced in the article on another page which seeks to overthrow settled beliefs on eccentric loading of structural members. Though at the start the author tries to appeal to preconceived notions by his ingenious title, "Is a Part Stronger Than the Whole?" yet he proceeds to make a strong and honest statement. He does not argue, but his illustrative examples have the force of argument.

However, it is important to recognize that the practical effect of the author's statements is little less than a complete ignoring of load eccentricity. This is a questionable proceeding, to put it mildly. True, there are cases—not a few, in fact—where the theoretical eccentricity is largely unreal. The diagonal brace whose connection plate is made eccentric by a single rivet will probably adjust itself to the loading with much lower overstress than the calculation indicates. A post set on an eccentric base block may crush down in such a way as to produce a fairly uniform base reaction. But under other conditions such a post may have a highly nonuniform reaction, and the post thereby be subjected to bending, which undoubtedly reduces the load-carrying capacity.

At bottom, the author's query is simply this: Does the stress in the extreme fiber measure the tax on the piece?

When the stress is tensile, is there any danger in initial cracks at the tensile extreme fiber? The author implies that there is not. Take a beam which has a narrow rib below its bottom flange, so proportioned that the beam would have a lesser extreme fiber stress if the rib were cut away; the claim is that when the projecting rib tears, the tear will stop at the main flange. The trouble with this statement is that it ignores the dangerous character of tension cracks and their great liability to extend progressively even at low stress. Broadly, a crack in a stressed piece of material is a sure forerunner of ultimate breakdown.

The author's argument is also misleading whenever adverse deflections are produced by nonuniformity of stress. In the case of a column, for instance, any one-sidedness of bearing means a deflection, and this increases the edge stress at the weak section of the column still further. The excess stress is not imaginary and is not harmless.

The essential warning, however, relates to the following: In steelwork we do not always deal with tough, ductile material. Local excess of stress, bending, eccentric loading, and initial cracks have a very different meaning for brittle or unsound steel than for normal steel. There are some structures standing today that have excessively high edge stresses at certain points. Usually the material is ductile and yields slightly, thereby relieving the local excess. But not all structural steel is ductile; some is brittle, and breaks rather than yield. The collapse of the Orpheum Theater should be remembered, where brittle plates failed at a point where high local bending existed.

Common-sense is called for in dealing with problems in eccentric stress. For instance, the question whether a column foundation resting on rock but carrying the column off center can distort enough to put any bending into the column is readily decided by the engineer's common-sense judgment. The entire subject calls less for definite rule than for the use of caution and common-sense together.

Ownership of Water Services

A favorite subject for discussion at water-works conventions for several decades has been who should own the service pipes. The question has come up again where so many questions both old and new are arising daily—before a state public-service commission. As no universal practice has yet been reached, and as light seems to be desired, an opportunity is afforded to point out anew what most water-works superintendents believe—namely, that service pipes, like the meters at their house ends, should be considered as a part of the water-works plant and supplied, laid, paid for and maintained by the city or company that owns the works.

Coming now to the case in point. The Montana Public Service Commission, as noted elsewhere in this issue, preliminary to issuing service regulations, asked the water utilities of the state why they should not own and care for the services.

The answer seems to have been that in Butte, for instance, the company would have to take over the 9,500 service lines now owned by the property holders at an added annual expense of $28,500 ($19,000 annual offset to depreciation and $9,500 for maintenance), and that it did not dare ask the consumers each to pay the $3 extra, to say nothing of $3 interest on eventual investment.

Nevertheless, the water consumers are actually giving up that $6. They are not paying it out year by year in cold cash, but each consumer is losing the interest on his $50 investment; he sees—or might see if he were wise enough—his $50 gradually wasting away, and he feels—or will soon feel—the need of expending another $50 for a new service line. In addition, he is paying out in real coin an average, say, of a dollar a year for repairs.

Those figures of investment and cost are predicated on individual action. As one of the Montana commissioners

in a dissenting opinion ably pointed out, the company, by centralized management and expert labor, could cut the first and annual costs of service lines by a third of what the individuals now pay. The company admits this is the case with meters and desires to keep the purchase and maintenance of such devices entirely in its own hands—as is quite proper. If the company owned the services, the rates might be raised to bring in $4 a year more revenue from each service line, and still the consumers would save $2 a year each.

Why should the water company fear to tell the users that fact and assume responsibility for the entire plant?

The language of the decision leaves possible the future readjustment of such matters in Montana. No valuations of these water-works have been made, apparently, and no scrutiny of rates as to their reasonableness. Probably if the commission should find the charges in any case unduly high by an amount about equal to the annual cost of the service lines under efficient company administration, taking the lines over from the individuals would be required in place of lowering the rate.

The experienced water-works man does not need to be told that utility ownership and maintenance of services are not only mutually advantageous to the utility and the consumer as a matter of finance, but are also necessary as a preventive of troublesome leaks or waste.

Cement for Pipe Joints

The use of portland cement for the joints of cast-iron water mains in southern California cities was described years ago in the columns of *Engineering News,* but has probably been forgotten by most of its readers. As stated in an article elsewhere in this issue, this innovation was not an isolated experiment to be tried and forgotten, as so many experiments are, but has become standard practice in a number of these cities and is defended not merely as a cheap substitute for the lead joint but as a better and more secure joint.

Why should the cement joint be absolutely watertight against an internal pressure of 100 lb. per sq.in. or more, while the cement joint for sewer pipes in practically universal use be exceedingly apt to leak under the ordinary pressure of ground water?

The answer to this question is probably to be sought in the manner in which the cement joint for water mains is made up—with a cement paste containing barely enough water to moisten it, packed into the pipe bell with a calking tool and hammered home as if it were lead.

It is common knowledge that the strength of a cement briquette is greatly affected by the packing of the cement in the briquette mold; but we recall no tests made to determine how strong a briquette could be produced by compressing a very dry paste under repeated hammer blows. The long experience with the cement water-pipe joint now available indicates that such cement is not only very strong but about as impermeable as metal.

The New Haven railroad bridge noted on page 1053 of this issue is described by the New York *Times,* Nov. 21, as follows: "The new bridge will have a capacity for eight tracks. Four tracks will be built immediately on the main structure and two on the superstructure, leaving room for two more on the superstructure when needed." Thus is a remarkable bridge accounted for.

Letters to the Editor

Waterproofing the Newark Terminal

Sir—The statement is made in the article in *Engineering News* of Oct. 28, describing the Newark Terminal, that "the waterproofing is unique in building construction because of its extent (500,000 sq.ft.) and because of its evident completeness." My recollection is that there was a larger area than this (about 56,000 sq.yd.) covered in the construction of the Boston South Terminal Station built nearly 20 yr. ago, which was designed to resist an upward water pressure of 500 lb. per sq.ft. and which has successfully served the purpose for which it was designed.

In the construction of the Boston Terminal 10 layers of felt and pitch were used. It would be interesting to know how the single layer of muslin used at Newark compares in cost, assuming it to be equally effective, as this will afford at least some measure by which progress in the art of waterproofing may be judged, although the conditions in Boston were probably more difficult than those in Newark.

F. Lavis.

50 Church St., New York City, Nov. 5, 1915.

[The designer of the waterproofing system used in the Newark Terminal has offered the following information.—Editor.]

Sir—Concerning the waterproofing at the Newark Terminal of the Public Service Ry., I beg to say that the cost was 16½c. per sq.ft. A portion of the structure was designed to withstand a maximum upward pressure of 950 lb., the normal upward water pressure being about 450 lb.

Oscar Sheffield,
Chief Engineer, Masonry Waterproofing Co.
307 Herkimer St., Brooklyn, N. Y.

Comparing Triple-Expansion and Centrifugal Pumps

Sir—In G. H. Gibson's article, "How Low Fixed Charges Favor Centrifugal Pumps," appearing in *Engineering News* of Nov. 4, 1915, I find there is an error as regards the cost of the Holly pump at Bissells Point. The price which he names was for two pumping units, instead of one. The cost of the unit referred to, including suction and discharge piping with two hydraulically operated gate valves, together with the bonus, was $103,953.

In making comparisons between the duties of pumps it should be stated how the heads on the units are measured. It is customary, in order to facilitate making a shop test, to measure the suction head or lift near the inlet flange and the discharge head near the outlet flange of a cen-

trifugal pump; then an allowance for difference between the velocity heads at these two points is made. In order that the shop test may be conducted without installing the auxiliaries to be furnished with the unit, it is also customary not to charge the steam used by the auxiliaries against the unit.

The auxiliaries (air pump, boiler-feed pump and air compressor) were an integral part of the Holly pump and were charged against it. The boiler-feed pump was required to deliver its full capacity against 200 lb. pressure, and this required over 4,500,000 ft.-lb. of work per hour. The friction losses in the suction pipe, gate valve and condenser were charged against the Holly pump. This was equivalent to a 2-ft. head.

There should be charged against a centrifugal pump the cost of the necessary increase in boiler capacity due to its lower duty. When a new building is required for a pumping station, the greater cost of the building necessary for the triple-expansion over that of the centrifugal should be charged to the triple. GEORGE M. PEEK,

Engineer in Charge of Construction.

St. Louis Water-Works, Nov. 8, 1915.

[Mr. Gibson has made the appended statement concerning Mr. Peek's correction and comments.—Editor.]

Sir—While the error regarding the cost of the Holly pump pointed out by Mr. Peek is regrettable, the De Laval pump still retains a considerable advantage in lower total annual cost per water horsepower, as will be seen after revising the figures in the last table of my article to take into account the correct price of the Holly pump:

	Centrifugal Unit	Reciprocating Unit
Cost of buildings and foundations, assumed	$20,000.00	$80,000.00
Cost of boilers, settings and stack assumed as $30 per nominal boiler horsepower	16,371.00	10,503.00
Annual interest, maintenance and depreciation charges on pumps and buildings	5,130.00	13,980.00
Annual interest, maintenance and depreciation charges on boilers	1,849.00	1,188.00
Total interest, repairs, maintenance and depreciation charges	6,979.00	15,168.00
Annual fuel cost of steam, at 13c. per 1,000,000 B.t.u., based on full load 24 hr. per day, 365 days per year	22,030.00	14,540.00
Total annual cost per water horsepower, exclusive of attendance and supplies, assuming 100% load factor	23.11	27.63

For 66.6% load factor, or full-load operation 16 hr. daily, the annual cost per water horsepower becomes $34.80 for the Holly pump, as against $25.90 for the De Laval pump.

The duty of the De Laval pump was calculated from the total head overcome by the pump—that is, the discharge head measured at the delivery nozzle plus the suction head at the suction nozzle, both corrected to the center line of the pump. As the two nozzles are of the same size, no correction for velocity was necessary.

Mr. Peek states that auxiliaries of the Holly pump, including air pump, boiler-feed pump and air compressor, are integral parts of the main unit and are operated by it. The air pump for the De Laval unit (of the common flywheel type) and the condensate pump (of the duplex type) are independent of the main unit, but the steam consumed by them was charged against the latter in figuring the duties stated in my article. At the same time, the exhaust from these units was delivered at atmospheric pressure to an open feed-water heater, instead of passing to the main condenser, and at least 80% of the heat represented by the live steam consumed by them was returned to the boilers in the feed water, raising the latter approximately 20° F. in temperature, with a fuel saving of nearly 2%. As the final temperature was still only 102° F., the exhaust of the independent boiler-feed pump can be utilized in the same way and the fuel cost for its operation will be practically negligible.

A turbine-driven boiler-feed pump would use not to exceed 600 lb. of steam per hour. If the pumping outfit were credited with heat returned in the condensate—as should be the case—the total net steam consumption charged to boiler feeding would not be more than the equivalent of approximately 90 lb. of steam per hour, which represents less than 0.5% in duty.

The larger boiler equipment required by the De Laval pump is fully taken into account in my article, where the nominal horsepower capacities of the boilers, upon which interest, depreciation, repairs, etc., are figured, are in each case based upon the respective steam consumptions in actual test, 30 lb. of steam per hour being taken as a boiler horsepower.

Figures were not at hand covering the costs of buildings and foundations and the cost of attendance in each case, so that the comparison of the two pumps is necessarily incomplete; but it is believed that the inclusion of these items would have resulted in a showing still more favorable to the steam-turbine-driven centrifugal pump.

GEORGE H. GIBSON.

Tribune Building, New York City, Nov. 15, 1915.

Popolopen Viaduct Design

Sir—I agree with "Concrete Engineer," whose letter on the Popolopen Creek viaduct was published on page 901 of *Engineering News* of Nov. 4, 1915. It would appear that the steel design might have been decided upon at first and an estimate on a concrete design asked for to stop comment. A concrete bridge to fit the location could have been designed and built for a very little more than the steel structure, unless foundations were very difficult.

I think a concrete bridge would have fitted the landscape better than the steel design. It was a pity there was only one competitor who submitted a design in concrete. ANOTHER CONCRETE ENGINEER.

Nashua, N. H., Nov. 6, 1915.

NOTES AND QUERIES

The Reinforcement of the Concrete Pavement under construction between Easton and Bethlehem, Penn., noted in "Engineering News" of Nov. 11, page 944, is not triangular-mesh wire but diamond-mesh wire made by the Trussed Concrete Steel Co., Detroit, Mich.

Deflection of Concrete Beams—A reader inquires what simple formula for deflection gives results which check with measurements on concrete beams. He says that the formulas published in some of the standard books are too complicated for practical use and do not give assurance of fitting the facts.

Offered to the Most Worthy Library—A complete file of "Engineering News" for about 15 yr., partly bound, is in the possession of N. S. Reeder, 24 Broad St., New York City, and is offered free to the most deserving applicant—schools, societies, libraries or individuals being given preference in that order.

Rebuilding Piers and Abutments, Black River Bridge

By Edward U. Smith*

The Wheeling & Lake Erie R.R. has recently completed rebuilding a girder bridge across the east branch of Black River on its Toledo division in Ohio, which required strengthening the substructure. The footings of the two river piers were enlarged in an original and effective way. Reinforced-concrete slab abutments were also built, as the old bridge, a curious combination of girders and timber trestle, had no abutments.

Thorough inspection of the piers showed that they were supported on oak piling and timber grillage, which were found to be in as good condition as when placed, 15 yr.

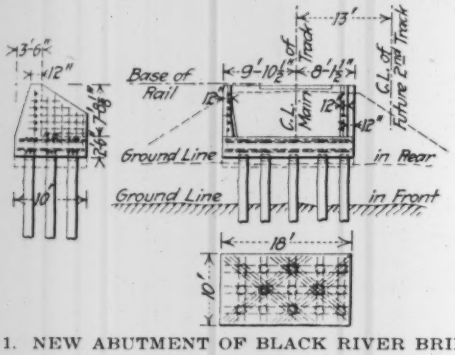


FIG. 1. NEW ABUTMENT OF BLACK RIVER BRIDGE

ago. The grillage beams were 24 in. on center, the spaces filled with concrete, making a concrete thickness of 3 ft. The top of the first course was at normal water level and the bottom of footing was about 4½ ft. below water. The stream is about 7 ft. deep. It was planned to substitute a new and complete plate-girder superstructure designed for heavier loading (Cooper's E-60), and this necessitated strengthening the pier foundations, although the pier shafts were adequate. It also meant building abutments to carry the end-span plate girders.

The plan adopted for reconstructing the pier footings was to drive two rows of mixed oak piling around and to a level below the bottom of the old footing, place a ring of concrete on top of the new piles and bring it up to the third course of stone, thus incasing the foundation and enlarging its bearing area, besides carrying the

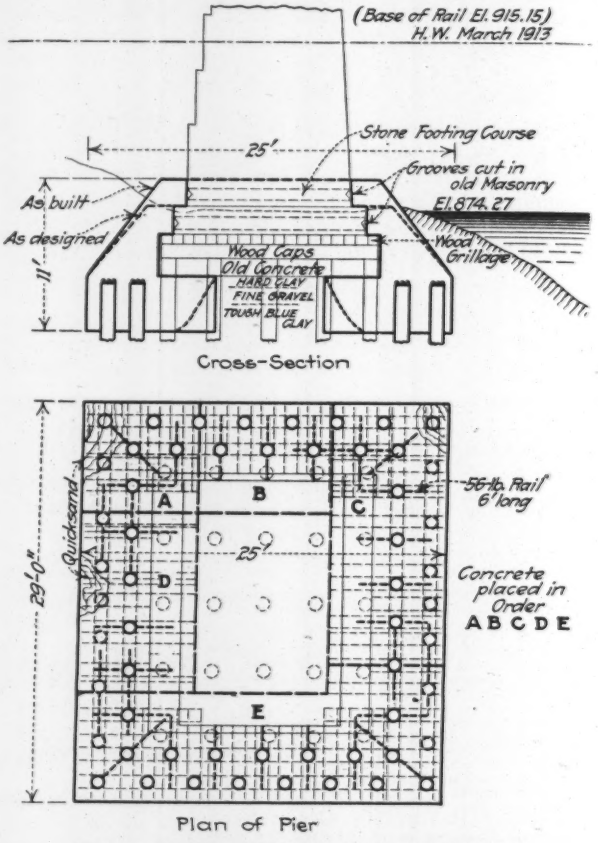


FIG. 4. PIER-FOOTING REINFORCEMENT

foundation deep enough to prevent future damage from scour. While the new concrete would not normally take the whole load of the pier, it was so designed as to be

FIGS. 2 AND 3. ENLARGING PIER FOOTINGS OF BLACK RIVER BRIDGE, W. & L. E. R.R.

*Assistant Engineer, Wheeling & Lake Erie R.R. Co., Electric Building, Cleveland, Ohio.

FIG. 5. CONCRETE PILES FOR WEST ABUTMENT

safe for the entire load if a slight settlement of the old work should take place.

In making the excavation (in the manner shown by the view Fig. 3) a bed of quicksand was struck. This made the work difficult, but called for no changes of method, and the reconstruction of the footing was successfully carried through. As shown by the drawing (Fig. 4) the concrete was carried up higher than originally intended. Radial reinforcement in the lower portion of the new footing concrete enables the extended footing to act as a corbel.

The abutments (Fig. 6) were designed as reinforced-concrete slabs carried on concrete piles. The piles chosen were those of the Great Lakes Dredge and Dock Co. They were octagonal, 16 in. by 40 ft., of uniform section, with cast-steel point and eight ½-in. vertical rods. They were driven by company forces, at a cost of 15c. per lin.ft. The work was done with an Ohio locomotive crane handling a 3-ton Vulcan steam hammer, which made 85 blows per minute. The piles were driven to refusal, most of them going down the full 40 ft., through gravel, clay and quicksand.

The slab over the piles, 10x18 ft., was made 36 in. thick for the east abutment and 54 in. thick for the west abutment. It has bands of reinforcing rods in both directions over the pile heads and also diagonal bands in two directions. The design of the slab was governed by punching shear, which required a thickness of 7½ in. over the piles.

The slab and back-wall reinforcement was arranged with a view to adding a future second track. The wing-wall which later will be between tracks and will serve as brace to the back-wall is straight, while the other is flared.

Steel was erected by a derrick car reaching out from the abutment end as shown in Fig. 7, picking up the girders from a point alongside the track (Fig. 6). The King Bridge Co., Cleveland, Ohio, fabricated the new steel, and the Ferro Construction Co., Chicago, erected it as subcontractor for the King Bridge Co.

After the girders were in place the abutments were backfilled with granulated slag. At the west end, where the piles projected several feet above the natural slope, the slag was rammed around and between the piles. At the east end the slope was such that it was necessary to excavate for the slab, so that only backfill had to be placed.

Cement Instead of Lead for Pipe Joints

The use of portland cement for making the joints of cast-iron water mains has become common in southern California cities. The practice appears to have originated in the water department of Long Beach, Calif., about ten years ago and the success there obtained has caused a number of other cities to adopt the same plan. The superintendent of the Long Beach water-works, Clark Shaw, declares that the cement joint is not merely cheaper than the ordinary lead joint, but is tighter and more permanent.

The method of making the joint is as follows: After the pipes are connected, a ring of dry jute is inserted in the bottom of the bell and the bell filled with neat cement paste. It is essential that this paste be mixed with only enough water to moisten it, but not enough to make it soft. After the bell is filled with the paste it is packed with a long calking tool, like that used for calking a lead joint. If the paste is of the right consistency it should pack under the tool until it is almost as hard as lead. (For an editorial discussion see page 1044).

FIGS. 6 AND 7. PLACING SUPERSTRUCTURE GIRDERS OF BLACK RIVER BRIDGE

After the first calking, the joint is refilled with paste, which is again packed solid with the calking iron; this is repeated a third time. The joint is then neatly rounded off with a collar of 1 : 1 cement and sand. The cement is allowed to set for 48 hr. before water pressure is turned on the pipe. In case of necessity, however, water may be turned into the pipe within 24 hr. The only lead joints now used in Long Beach are in pipe which has to be put in service as soon as it is laid. The pipe used has the standard hub and spigot. A line of pipe 6,400 ft. long was recently laid with cement joints and not a single leak was found when it was tested.

When it is necessary to take up pipe laid with the cement joint, a section of the cement on top, about 60° wide, is chipped out to the bottom of the bell, and the joint is then readily broken. This work is much more easily done than unjointing pipe having lead joints.

An important advantage of the cement-jointed pipe is that the conductivity of the line for electric currents is much reduced, as there is cement insulation at every joint. Liability of damage to the line by electrolysis is greatly reduced, although if one joint or only a few were made with cement there might be corrosion at these points.

A comparison of costs of the material required for a 4-in. pipe joint shows that with lead at 5½c. per lb. the material for a lead joint cost 29c., and the material for a cement joint 3½c. At the present high prices of lead the difference is much greater, and on larger sizes of pipe the large saving possible makes the change one which is well worth the attention of water-works engineers. There is also much less expense for labor in the cement joint than for the lead joint, and the work is simpler.

Alternate Specifications for Public Work Are Legal

By Daniel T. Pierce*

No little attention is being given to the subject of alternate specifications and bidding from the viewpoint of public policy. The reason why this subject is always more or less alive is that the common requirement that a low bid under a general specification must be accepted is productive of so much poor construction. Recourse is had to an increasing extent to alternate specifications, separate divisions of which may describe entirely different materials or types of construction or different kinds of the same type of materials, the authorities reserving the right to accept the lowest bid under any division of the specification.

The alternate specifications avoid the undesirability of "open specifications," which confer a monopoly upon mere cheapness; of "closed specifications," which may be considered objectionable because they may create monopoly, and enable the purchaser to consider quality as well as price. While this is conceded, there is raised from time to time the objection that alternate specifications are not legal under statutory requirements that public contracts must be awarded to the lowest bidder. The following review of decisions on this point shows that the legality of the alternate specification has been upheld under a variety of conditions in at least nine states.

*Executive Assistant, Barber Asphalt Paving Co., Philadelphia, Penn.

The case of the City of Connersville vs. Merrill, an Indiana case (42 Northeastern, 1112), arose under an Indiana statute requiring sidewalk-improvement contracts to be let upon competitive bidding to the lowest responsible bidder, and provided that such improvements could be made pursuant to an ordinance which was required to state "the kind, size and location of the proposed improvement."

The City Council adopted a sidewalk-improvement ordinance which provided that certain sidewalks "should be made of either free or limestone flagging or of artificial cement stone as the Common Council might determine on receipt of bids therefor." Separate bids were received for the different materials specified and a contract was awarded on one of such bids. The validity of the contract was called in question by an abutting property owner. The court held that the contract was valid.

> There was no uncertainty in the material to be used, so far as the bidders were concerned. Thus bidders were put upon an equality, and the city and the property owners had the benefit of competition on each brand or special kind of the general material to be used. They were invited to submit bids for the improvement on three different kinds of material. The bids were made on this basis, and the lowest and best bid for either kind of improvement—that is to say, the lowest bid for artificial cement stone was accepted.

The case of Dixey vs. Atlantic City, a New Jersey case (58 Atlantic, 370), likewise upholds the principle of alternative specifications. Atlantic City advertised for bids for paving certain streets with vitrified blocks. The Delaware River Quarry and Construction Co. submitted a bid specifying various separate prices for the work, according to the particular type or brand of block to be used. One item of this bid was accepted and a contract let pursuant thereto. Certain taxpayers filed a suit calling in question the validity of the contract on the ground that the successful bidder had no right to submit alternative bids, as this would be in violation of the statute requiring competitive bidding. The court held the contract valid, declaring:

> The bid of the successful bidder gave the city the advantage of knowing just what were high and what were low-priced blocks, and we think the specifications did not limit bidders to naming blocks of only one price, but permitted them to bid for different kinds of blocks at different prices, and if they were able to furnish more than one kind of block at the same price, they might do that, also submitting samples and the information as to manufacturer required.

Case of the Philadelphia Filtration Plant

In the case of Parker vs. City of Philadelphia, a Pennsylvania case (69 Atlantic, 670), alternate specifications were again upheld. Philadelphia advertised for bids for a filtration plant. One item of the specifications called for controllers to measure the output from the filters. Six different types of controllers were specified, leaving it to the city to determine, after the receipt of bids, which type should be adopted. Alternative bids were requested and submitted upon several different types. A taxpayer sought to impeach the validity of the contract, but the court held the contract legal. A part of the decision follows:

> Where the thing to be furnished can be manufactured by anyone in precise conformity to certain and specific requirements of the specifications, or purchased in an open market at a standard price, there is no valid reason for asking for alternative bids; but where the thing is protected by a patent or cannot be bought in an open market, and any one of a number of particular kinds fully complies with the established standard, the interest of a municipality will be best served by asking for bids for any or all. The field is equally

open to each bidder; the power of a monopoly in the thing is broken. The general custom of long standing has been to require bidders in certain classes of public work to furnish samples of the articles they proposed to furnish in order that the quality of the sample may be taken into consideration in awarding the contract, thus bringing into competition both the price and the quality of the thing to be furnished.

Farther on in the opinion it continues as follows:

We are of the opinion that under our law specifications for alternative bids are not per se unlawful.

Cases Involving Various Types of Pavement

The case of Ex Parte City of Paducah, a Kentucky case (89 Southwestern, 302), is one where the city adopted specifications and advertised for bids for the improvement of a street. The specifications provided that the street should be improved with vitrified brick, bitulithic or bituminous macadam. Alternative bids were requested and submitted on these several types of pavement. The court held the contract for bitulithic valid, saying:

Nor are the ordinances objectionable for the reason contended for by counsel for the city, on account of different materials being mentioned therein with which the streets were to be constructed. It is obviously good policy, and to the best interest of the public, that ordinances of this nature should be so framed as to bring about competition in the bidding of such work.

The case of Campbell vs. Southern Bitulithic Co., a Kentucky case (106 Southwestern, 1189), arose when the City of Ashland adopted specifications for paving with vitrified brick or bitulithic pavement, as might be determined after receipt of bids. Alternative bids were advertised for and received upon both types of pavement. The brick bids were lower than the bitulithic bids. A contract was awarded to the lowest responsible bidder on bitulithic. A suit was brought to enjoin the performance of the contract on the ground that the contract was let in violation of the statute requiring such contracts to be let upon competitive bidding to the lowest bidder. The court held the contract valid and quoted with approval the Paducah case cited above.

In the case of Emmeret vs. Elyria, an Ohio case (78 Northwestern, 269), the city required paving contracts to be let upon competitive bidding and provided that improvement ordinances should contain a statement of the general nature of the improvement and the character of the materials thereof. The city adopted an ordinance and advertised for bids for paving a certain street with brick or asphalt or other material as might thereafter be determined. After receipt of alternative bids a contract was awarded for asphalt. A property owner sought to enjoin the performance of the contract, but the court upheld it.

It appears from the finding of facts that council determined that the paving material should be asphalt, brick or other material, as might thereafter be determined. This meets the requirements of the statute. Prior to the adoption of the present code it was provided, as to some cities, that the kind of materials should not be determined until after bids had been received, the reason being that it promoted competition and tended to prevent collusion among bidders.

State of Michigan vs. City of Detroit

The case of Attorney-General vs. City of Detroit, a Michigan case (26 Michigan, 263), arose when the City of Detroit adopted specifications and advertised for bids to pave certain streets. The specifications and advertisement called for alternative bids upon several kinds of wood and stone pavement. After the receipt of alternative bids a contract was awarded for the Ballard wooden-block pavement. The validity of the contract was questioned by the attorney-general in this case, but the court held the contract valid in an elaborate opinion by Judge Cooley. This case is the earliest and may probably be regarded as the leading case on the right of municipalities to adopt alternative specifications.

Baltimore Paving Specifications Upheld

In the case of Mayor of Baltimore vs. Flack, a Maryland case (64 Atlantic, 702), the city advertised for bids for the improvement of certain streets with asphalt, vitrified brick or bitulithic, as might be determined after receipt of bids. Alternative bids were submitted upon the several types of pavement and a contract was awarded to the lowest responsible bidder on bitulithic. Property owners sought to enjoin the performance of the contract on the ground that alternative bidding was not permissible under the Maryland statute which required an award to the lowest responsible bidder. The court held the contract valid in an elaborate and carefully considered opinion, containing these statements:

Thus competition was invited and secured both as to materials and as to price, and the contract was awarded to the lowest responsible bidder on the material selected, though there was another bidder whose price was lower on a different material. Were this method of procedure and this action in violation of Sections 14 and 15 of the charter? There are two kinds of competition—the one, competition between different things which will equally answer the same general purpose; and the other, competition between the prices bid respectively upon each of those distinct things. What difference is there, or can there be, looking solely to the extent of that authority, between selecting the kind of pavement before bids are asked for and selecting the kind of pavement after the bids have been received and opened, upon distinct sets of specifications descriptive of wholly different kinds of pavements, but all of which are suited to the same general purpose? Manifestly, none whatever; though in some particulars, not touching the power itself, but affecting the ultimate result in other ways, there may be a difference between the two methods. Thus by the selection of the materials or the kind of pavement, after the bids have been received, combinations between bidders to inflate prices may be in a great measure avoided, since it is altogether improbable that parties who compete for the adoption of their respective materials will all ask exorbitant prices, as each party will most likely strive by depressing prices to secure the contract. As in such an instance no bidder knows in advance which kind of pavement will be selected, each is in reality stimulated to propose terms which in order to secure him the contract will produce the best results so far as the public are concerned.

New York Decisions on Use of Patented Materials

We now come to the case of Warren Brothers Co. vs. New York City (190 New York Reports, 299). This shows that the general principle set forth in the foregoing has likewise been followed in New York State. In this case the Court of Appeals reversed the Appellate Division, which had held to the contrary. Section 1554 of the New York Charter provided as follows:

Except for repairs, no patented pavement shall be laid and no patented article shall be advertised for, contracted for or purchased, except under such circumstances that there can be a fair and reasonable opportunity for competition, the conditions to secure which shall be prescribed by the Board of Estimate and Apportionment.

Under this section bids were asked for asphalt blocks, sheet asphalt and bitulithic. The question in the case was whether under such circumstances, where bids were asked for three different kinds of pavement, there could be a fair and reasonable opportunity for competition within the meaning of the charter provision just quoted. The Court of Appeals reversed the Appellate Division and held that there could be fair and reasonable opportunity for competition.

We are of opinion that the scheme devised by the Board of Estimate and Apportionment permitting the owners of patented and unpatented pavements to join in bidding for contracts to lay the same under section 1554 of the charter is, under all the circumstances, feasible, workable and affords a fair and reasonable opportunity for competition under said section.

The case of Warren Brothers Co. vs. City of New York was likewise cited with approval and followed by the Appellate Division in New York in the case of Holly vs. City of New York (128 N. Y. App. Div. Reps., 502), where bids were taken on four different types of pipe. We quote from this decision as follows:

Bids were advertised for four types of pipe on what we can only conclude were equal terms, and if it be assumed that type D is a patented article and the subject of monopoly, the conditions were similar to those approved by the court in the case just cited, except that the conditions were not prescribed by the Board of Estimate and Apportionment, which I do not think enters into the question in the present case, for the reasons already stated. That case related to bids for patented pavements, and the court said: "The dictates of public policy under the circumstances here presented would seem to require that the owners of patented and unpatented pavements should bid on equal terms."

The foregoing cases show that the rule is general that alternative specifications calling for two or more materials of a different kind or quality are not invalid because bids are taken under statutes requiring the contract to be let to the lowest responsible bidder. On the contrary, the courts have held that by taking alternative bids on different kinds and qualities, not only is competition secured in price, but is likewise secured in quality and prevents collusion.

Fifty-Ton Testing Machine Built in College Shop

By Curtis C. Myers*

Conditions often arise in making investigations where it is desirable to use special testing machines for a short time in order to secure data for reports which are being worked up by a committee. It is often impossible to invest the requisite money for permanent equipment and yet the data are necessary for a convincing report.

This condition arose recently with a committee of the Engineers' Club of Cincinnati when it was desired to make some reinforced-concrete T-beams 10 ft. long by 12 in. wide and 12 in. deep.

A testing machine was wanted which would fill the following requirements:

(1) A portable testing machine of 100,000 lb. capacity which may be used under a hatchway where an overhead crane was available for handling the beams. It was not desirable to locate the machine permanently in this position, as it was on a driveway which was used frequently. (2) A machine which would not require a pit in the floor to accommonate the weighing or loading mechanism. (3) A machine which could be built in a repair shop having ordinary shop equipment. (4) A machine not to cost over $250. This limit made necessary a design free from castings or other details which would be costly.

*In charge of Cement Laboratory, University of Cincinnati, Cincinnati, Ohio.

The transverse testing machines on the market do not meet all those requirements, as the large size require a permanent location and a pit for their loading mechanism, and their capacity is not great enough for a 42-in. beam. The machine here described was built for $245 and has given satisfaction. It was built almost wholly of scrap material, as the structural members were picked up in local junk yards, with the exception of the heavy 6-in. I-beams for the main weighing levers. The weighing mechanism was made of a discarded long-beam briquette-testing machine and the loading was accomplished with a 70-ton screw jack. The weighing mechanism could be replaced by a platform scale and the jack could be a hydraulic jack, with perhaps a little more compactness.

The machine was designed for a safe load of 100,000 lb., but was loaded to 146,000 lb. in the tests with no failure. After calibration a load of 150,000 lb. was applied and accurately measured.

While designed for beam testing, the machine may be used to test columns up to 8 ft. long and also for other compression tests, like concrete culverts. If desired, tension tests of limited range might be made.

Method of Construction

The machine was built in the University of Cincinnati repair shop by one of the students. There was no serious trouble encountered in the building, which could not be handled in any ordinary repair shop with limited equipment. The main I-beams were used as found and but few extra holes were drilled in them. It was appreciated that longer beams could have been used so that the machine would have more range simply by moving the weighing end. The short beams were cut to length by an oxyacetylene torch, this method proving cheaper than drilling. All members were bolted together, as there were no facilities for riveting. In addition, bolts were desirable in order to permit dismantling in case of storage.

The general details are shown in Fig. 1. The wheels were discarded mixer drum rollers. The main transverse I-beams have webs stiffened with angles, and a 32-in. 70-lb. rail is bolted to the top of the highest of these beams. This rail acts as one of the knife edges for supporting the beam to be tested. The tension rods were made of discarded bridge rods and the heads were welded on of the same material. The plates holding these rods were made 1½ in. thick, and they proved heavy enough.

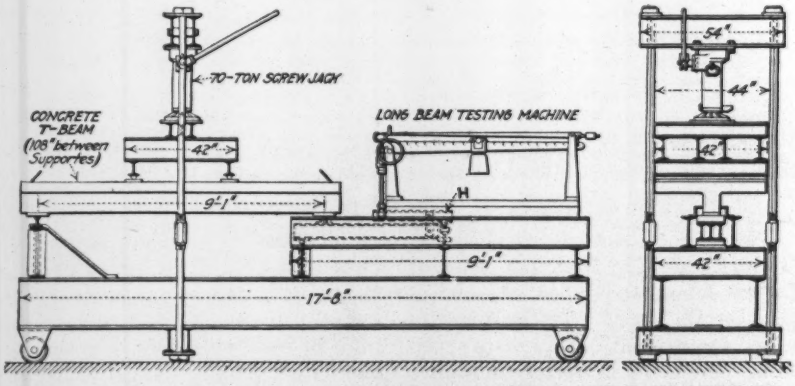


FIG. 1. DETAILS OF 100,000-LB. TESTING MACHINE BUILT IN SHOP OF UNIVERSITY OF CINCINNATI

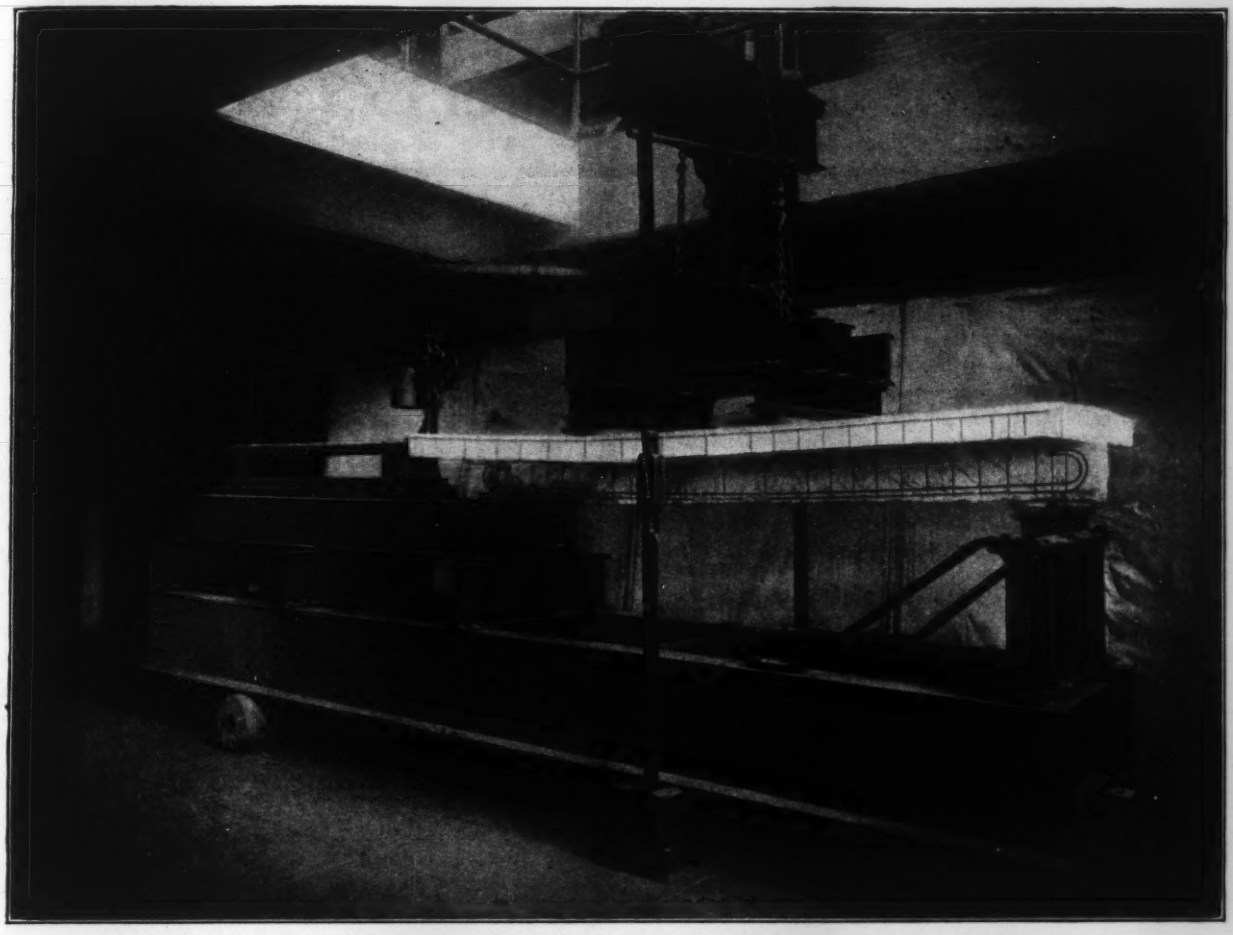


FIG. 2. VIEW OF MACHINE WITH CONCRETE BEAM IN COMPRESSION TEST

The knife edges and bearings were made of tool steel hardened in fish oil, and the temper was not so high as to cause any trouble from breaking the edges, which were rounded slightly with an oil stone. The 1¼-in. pins through the links and the links themselves were case-hardened, and both gave satisfactory service.

The ball-bearing screw jack was bolted to its bed plate and fastened with clamps and U-bolts to the top plate. This arrangement made possible the handling of the whole upper portion by the overhead crane. The long-beam testing machine was connected to the lever *H* by an eye-bolt fastened into a clevis with hardened bearings. The adjustment at the top end was made by two bolts which connected to a second clevis on the end of the scale beam.

Operation of the Machine

The method of operation when a beam was to be tested was as follows: The two tension rods were screwed out of the turnbuckles and a crane lifted the whole top part up and to one side. The beam was then lifted by means of eye-bars in the ends. Bearing plates were now adjusted on the knife edges and the beam placed in position. The top header was set on the beam and the turnbuckles were adjusted. This operation of taking off the top portion, adjusting the beam and assembling the machine took about 20 min. All the beams tested were reinforced, and there was no trouble from sudden failures. If a beam failed suddenly there would be danger of the whole top portion dropping down, and this would have to be provided for by some kind of a buffer to restrain the header. Then after the rods were screwed back into the turnbuckles a platform was placed in position and two men easily applied the load as wanted. The cost of the machine is as follows:

Old material	$68.90
New material	21.40
Bolts	11.20
Welding rods	11.00
Oxyacetylene cutting	7.00
Labor	125.50
Total	$245.00

The First Commercial Oil-Burning Tug in New York Harbor is the "Mexpet," which went into service this summer. It is owned by the Mexican Petroleum Co. and is used for handling oil tankers and ocean vessels docking at the company's pier to fill its bunkers with fuel oil. The tug is 112 ft. overall and has a beam of 24½ ft. The hold is 14 ft. deep. The rudder is unusually large for ease in short turning. The boat has one Scotch boiler 14½ ft. in diameter by 12 ft. long, with three corrugated suspension furnaces 48 in. in diameter. The pressure carried is 165 lb. per sq.in. The mechanical fuel-oil firing system of W. A. White is employed, there being two oil pumps and two heaters. The main engine is an inverted compound with 18- and 38-in. cylinders and a 26-in. stroke. In service the "Mexpet" has evaporated 16.4 lb. of water per pound of oil from and at 212° F. Oil is sprayed at from 240° to 280° F. under 40- to 80-lb. pressure. The engine horsepower ranges from 650 to 750, and a speed of 10 knots has been developed. Ninety tons of fuel oil is carried, sufficient for 10 days' run. A crew of 10 men is required.

News of the Engineering World

Important Stream-Pollution Decisions in New Jersey

The right of the State Department of Health of New Jersey to compel a municipality on the Delaware River to cease polluting that stream by sewage, notwithstanding the fact that it is an interstate river and being polluted by Pennsylvania municipalities, was affirmed by the Court of Errors and Appeals in a decision handed down on Nov. 15 in the long-pending suit between the state and Phillipsburg. The court also held valid the transfer to the State Board of Health of 1908 of the powers of the State Sewage Commission created in 1900. (The board was succeeded in turn by the present department on July 1, 1915.)

Another important decision rendered by the same court on the same day was one that materially affects Newark and other municipalities taking water from the Passaic River for municipal purposes. This suit was brought by dyeing concerns in Paterson, who sued, as riparian owners, for damages for the diversion of water from the river. The court held that a riparian owner who was polluting the stream to the injury of the people of the state could not collect substantial damages for diversion of water for public use.

The Maltrata Railway Accident

A railway accident which in number of casualties is one of the worst on record occurred Oct. 5 on the Mexican Ry. Details have been secured with difficulty. This railway is described elsewhere in this issue. The accident occurred on the Maltrata incline (between Orizaba and Boca del Monte), where the line makes an ascent of 4,000 ft. in 25 mi., with a general grade of 4% and some stretches of 5%. The alignment is very crooked and has numerous 16° curves, as shown by the plan in another column.

The accident was near Alta Luz, at tunnel No. 13, about 8 mi. west of Maltrata. The tunnel is 1 mi. distant from and 1,600 ft. above Alta Luz. The morning passenger train from Vera Cruz to Mexico was ascending the grade, hauled by a Fairlie duplex locomotive. In the tunnel the coupling to the engine gave way and the engine ran ahead, the separation of the air hose setting the brakes on the cars. The engineman did not at once realize what had happened, but after running a short distance he stopped and backed up, apparently without taking proper precautions. The tunnel is on a sharp curve, and in the darkness he miscalculated the position of the train and bumped into it with such force as to start it down the grade, the air brakes being in some way released.

The train is said to have run about 4 mi. in 2 min. before it was derailed, and then ran off the roadbed and plunged down the side of the mountain. Attempts were made to set the handbrakes, but with the panic and the crowding of passengers upon the platforms the brakemen were not able to reach the brake handles. Nearly all those on the train were killed or injured. The number of casualties is estimated at 300, a great majority of these being fatal. All the passengers who jumped from the train were killed. The bodies were collected and cremated at the side of the track, thus obliterating all traces of the dead as well as concealing the actual numbers. The only foreigners killed were a Frenchman and his wife, but their two children were unhurt.

The engineman and train crew were Mexicans. We are informed that the road has for some time been in the hands of the Carranza forces, the English management having no voice in its operation and the operation being of a haphazard character. The local papers and the brief dispatches sent out minimized the severity of the accident, but the foregoing particulars have been secured from a reliable source.

Death of Col. Francis G. Ward

Col. Francis G. Ward, for the last fifteen years Commissioner of Public Works of Buffalo, N. Y., died Nov. 16 at his home in that city. He had been ill for over a year.

He was born Mar. 8, 1856, in Cayuga County, New York, and was a lineal descendant of Col. John Ward, a soldier of the American revolution. He was educated at private schools in this country and in France and studied engineering at L'Ecole des Ponts et Chausses in Paris.

In 1875 Mr. Ward went to Buffalo as agent for the Laflin & Rand Powder Co. Two years later he entered the service of the New York Central & Hudson River R.R. as a yardmaster, being subsequently promoted to the positions of dispatcher and assistant superintendent of the Grand Central Terminal, New York City.

After ten years of varied experience in railway work in this country he received the appointment of superintendent of the Panama R.R. and for two years lived on the Isthmus of Panama. In 1887 he was transferred to Paris as manager of the general offices of the Panama R.R.

His work at Panama and in Paris attracted the attention of the French Government, and in 1888 he was employed to take charge of important surveys for proposed railways in Asiatic Turkey. Subsequently he had charge of dock construction in Algiers.

Mr. Ward returned to Buffalo in 1890, and after an unsuccessful venture in the soap business he began his public career as the chief of the city bureau of water in 1896. His service here was interrupted at the outbreak of the Spanish-American War, when he went to Cuba as Lieutenant-Colonel of the 202d New York Volunteers.

In 1900 the Buffalo city charter was changed so as to place the department of public works under a single commissioner, and Colonel Ward was elected to that office. He was reëlected for two- and four-year terms through

five successive city administrations—a remarkable record in an elective municipal office.

The department of public works being the largest city bureau, employing about 3,000 persons, has been a continual target for the city politicians, and Colonel Ward's organization was from time to time made the subject of much criticism and many investigations. The fact that he was continuously retained in office by popular election is sufficient vindication of the work he accomplished.

All his activities were not devoted to the narrow field of technical work. He was a linguist of unusual ability and could read and talk most of the European languages, and was a diligent student of history and philosophy.

He is survived by a widow, two sons and two married daughters.

Seattle City Engineer Reports on Cedar Lake Reservoir

City Engineer A. H. Dimock of Seattle, Wash., has submitted under date of Nov. 8, 1915, a report of his studies to find some means by which to render the Cedar Lake storage reservoir of the Seattle water-supply and power development available for use. As described in *Engineering News* of Jan. 21, and Apr. 15, 1915, the great concrete dam (160 ft. high and 795 ft. long on the crest) recently completed at a cost of $1,500,000 is practically useless because of the leakage under a moranial bank into which one end of the dam projects.

Analyses of samples of materials taken from various depths in pits sunk through the leaky bank show that it is composed largely of coarse sand and gravel with a few intervening strata of finer materials. The level of the water in these pits fluctuates with the water level of the reservoir, eliminating all doubt of the porosity of the material.

Experiments have been made to see whether or not a silting process could be devised which would utilize nearby soil; but the only material available for silt is a fine sand and surface soil, with which it was found impracticable to make an impervious mixture.

Two methods are proposed to reduce the leakage to a reasonable amount. One is the construction of a curtain wall (see *Engineering News* of Apr. 15, 1915); but this method Mr. Dimock is inclined to consider impracticable. The other method is to cover the porous area with a mat or layer of impervious material, a mixture of the local sand and gravel with some other material, the nature of which is not disclosed. Experiments, however, have convinced the city engineer that it is practicable to make such a mixture.

If this method is adopted it is proposed as a preliminary to remove all logs, stumps and other débris from the reservoir bottom, smooth off all slopes to 3 on 1 and utilize the excavated material to fill the old river channel. This work would be done under one contract, and if filling the river bed stopped the leakage, the second contract for covering the reservoir bottom would not be entered into. There is about 150 acres to be treated, and the cost of clearing and smoothing the slopes is estimated at $300,000. The cost of the impervious coating is stated to "amount to a considerable sum of money."

Mr. Dimock recommends caution, and he believes the scheme should be tried out on 10 or 15 acres before proceeding further. He believes that "it is entirely feasible and practicable to secure a reservoir which will retain the impounded water." In view of the unusual problems and the large interests involved, he has asked to be authorized to secure such expert advice and assistance as may be necessary to work out the details.

New Haven Road to Build New Thames River Bridge

A new Thames River bridge is to be built at New London, Conn., by the New York, New Haven & Hartford R.R. The present bridge, which consists of five spans (150 ft., 310 ft., 503-ft. draw span, 310 ft. and 150 ft.), was built in 1888-89 and originally was a two-track structure. In 1908, owing to increased train weights, it was necessary to limit the use of the bridge to single-track operation. The bridge is on the main line of the railway between New York City and Providence and Boston, so that the reduced capacity of the structure has seriously hampered the efficiency of the whole line. The new bridge is to have a substructure capable of carrying four tracks with the heaviest known loading; but the superstructure to be first built will be for two tracks only. It will also be a five-span structure with an electrically operated bascule lift span. There are to be three spans of 330 ft., one of 185 ft. and one of 212 ft. The new bridge is to be 185 ft. upstream from the present structure, which is to be presented to the State of Connecticut for a highway bridge. The four piers for the new bridge will require foundations ranging from 75 to 142 ft. deep. The piers are to be of concrete, with masonry facing between high and low water; three will be constructed by open-well dredging and a pneumatic caisson will be used for the fourth. The extreme dimensions of the piers are 42x98 ft. About 48,000 cu.yd. of concrete will be required. The superstructure will contain 5,000 tons of steel. The work includes two bridge approaches of 1,300 and 1,600 ft. respectively. Contracts have not yet been let, but it is assumed that the work will be completed in 1917. The plans have been prepared by the engineering department of the New York, New Haven & Hartford R.R., under the direction of Edward Gagel, Chief Engineer; I. D. Waterman, Construction Engineer, and W. H. Moore, Bridge Engineer. The following are the consulting engineers: Gustav Lindenthal, E. H. McHenry, Ralph Mojeski and J. P. Snow. The estimated cost is $2,500,000.

Irrigation by Los Angeles Aqueduct Water

Further progress in making ready to utilize the surplus water of the Los Angeles Aqueduct will result from the approval by the California Supreme Court of the procedure for the formation of the Los Angeles Water District No. 3. This district comprises 77,000 acres of land in the San Fernando Valley, which has recently been annexed to Los Angeles for the purpose of securing the use of the water named. The district proposes to issue $2,600,000 of 6% bonds, half of which it intends to put on sale shortly.

The recently enacted "County Water District Law," under which the district previously named was organized, has itself been approved by the State Supreme Court.

The law provides that upon petition to the Board of Supervisors of the County from territory that is partly or wholly unincorporated, the supervisors shall proceed with the organization of a water district, either for domestic- or irrigation-water service, and that after the district is organized the county officials shall act as the officers of the district, collecting the taxes of the district for interest and principal on the bonds, the same as regular county taxes, and eliminating entirely the local district officers, who in years gone by have caused much trouble in irrigation districts.

The law further provides that when all of the territory so served shall become annexed to a city having a municipal water-works, these county irrigation district works shall automatically become a portion of the municipal water-works. The city officials are then substituted in place of the county officials, and the taxes for interest and principal on the bonds are collected as a portion of the regular city taxes. These bonds, therefore, are quasi-county or municipal bonds and should have a market value about the same as school, county or city bonds.

In the case of the district under consideration it is expected that eventually the irrigated area will be built up and all the water formerly used for irrigation will be required for domestic and other municipal purposes.

Court Decision Adverse to Luten Bridge Patents

A decision against the validity of seven patents on reinforced-concrete bridge construction issued to Daniel B. Luten was handed down Nov. 18 by Judge R. E. Lewis in the United States District Court at Denver, Colo. In the case decided, Daniel B. Luten sued Washburn and others for infringement of the following patents: Nos. 852,970; 853,183; 853,202; 853,203, dated May 7, 1907; 933,771, dated Sept. 14, 1909; 979,776, dated Dec. 27, 1910, was 989,272, dated Apr. 11, 1911.

Washburn was a contractor who built a 30-ft. span reinforced-concrete bridge for a Colorado county. The county joined in the defense of the suit for infringement and employed Prof. M. S. Ketchum as expert in the case. Among numerous patents cited to the court by the expert as anticipating the alleged inventions of Luten was a patent to Coignet, No. 88,457, issued in 1869, containing the following claim: "The introduction in the body of artificial stones or in the body of artificial stone monolithic structures made of agglomerated stone paste of skeletons or metallic framework, linked or arranged so as to strength the same."

The expert for the complainant disclaimed that the patents in the suit embodied anything beyond placing the steel in a new way that produced better results in a more efficient form.

Judge Lewis in his decision declared that in placing the steel in position to conform to certain mechanical principles, in the patents in suit, Luten had in each instance made application of mere mechanical skill and that what he did was in no sense the work of inventive genius.

The judge refused to consider as binding the previous decisions rendered in favor of the Luten patents, holding that the evidence showed that in all these decisions the cases for the defense had not been adequately presented.

The Joint Sewage-Disposal Project for Pasadena, South Pasadena and Alhambra, Calif., has progressed as far as the purchase, on Nov. 10, of 600 acres of land for a disposal site at a reported cost of $150,000.

A Highway Engineer is to be engaged by the State Highway Commission of Florida at its meeting on Dec. 1, at which time the work of the commission will be organized. The chairman is W. J. Hillman, of Live Oak, Fla.

A 600-Mile Oil Line is contemplated to run between Casper, Wyo., and Omaha, Neb. Governor Kendrick, of Wyoming, has appointed a committee to decide upon the feasibility of the project, which is expected not to exceed in cost $3,000,000.

A Car Ferry for the Canadian Northern Ry. is to be operated between Vancouver, B. C., and Patrica Bay on Vancouver Island. From the latter point a railway will be built to the City of Victoria, a distance of 16 mi. Two car-ferry steamers have been ordered for this service, which will enable passengers to travel to and from the Port of Victoria without change of cars.

A Saving of $100,000 per Year by cooling the water used in the condensing tanks of the garbage plants of the Pacific Reduction Co. in Los Angeles is announced in the annual report of Homer Hamlin, City Engineer. The city furnishes the water for this reduction plant, and the supply needed has been cut down by the installation of cooling tower and 18x22x10-ft. storage reservoir. The cooling tower is 4 ft. wide, 100 ft. long and 36 ft. above ground.

Erie Adopts Flood-Prevention Plan—The City Council of Erie, Penn., has adopted formally the flood-prevention plan recommended by Farley Gannett, as reported Nov. 11, 1915, p. 937. The channel of Mill Creek through the city will be converted into a horseshoe-shaped reinforced-concrete conduit 22x18 ft., with a flow capacity of 12,000 cu.ft. per sec. This is slightly more than the peak flow of the Aug. 3 flood. No storage is to be provided on the stream. The total cost will be $900,000. Work can be started without a referendum vote.

Recovering Benzol from Coal Gas is a feature of the byproduct coke-oven gas plant of the Citizens Gas Co. of Indianapolis, Ind. This step was taken owing to the necessity of scrubbing the gas to remove naphthalene, which caused trouble by clogging the mains. By scrubbing the gas the light oil is recovered and carries with it the objectionable naphthalene. By refining the light oil, benzol is obtained, and at the present time is commanding a high price. A complete refining plant for this purpose has been installed, and though a considerable percentage of the benzol must be returned to the gas for enrichment, there is a certain amount available for sale.

A Conference of Iowa Sewage-Works Operators was held at Ames, Iowa, Nov. 2 and 3, under the coöperative auspices of the Engineering Division of Iowa State College and the Iowa State Board of Health. Over twenty operators were in attendance, besides representatives of the college and of the board, several men from outside the state and a number of engineering students. The 21 topics on the carefully arranged program were designed to give a bird's-eye view not merely of the various methods of sewage treatment, but also of the elements of each method and of the best way of meeting operating problems. Mimeographed notes on the work of the conference were distributed to those present. Visits were made to three sewage works—those of the college, the city and Nevada, Iowa.

Cracks in a Reservoir Lining at Aberdeen, Wash., have been investigated by R. H. Thomson, consulting engineer, Seattle. Mr. Thomson states that the cracks are not due to any fault in the reinforced-concrete lining (built without expansion joints, as described on page 1034 of this issue), but are caused by movement of the earth fill, which forms half the depth of the reservoir at the place where the cracks have occurred. Of the few cracks in the lining, Mr. Thomson states that the widest is probably ¼ in. across at the surface, and that L. D. Kelsey, city engineer, has taken needful measures by making "insurance applications" in the form of burlap strips soaked with asphalt to prevent leakage into the embankment. Subsequent advices from C. E. Fowler, consulting engineer, Seattle, indicate that the earth fill slid on the rock beneath it.

The Eugenia Falls Plant of the Ontario Hydro-Electric System was officially opened on Nov. 18 by Mayor J. McQuaker, of Owen Sound, in the presence of the representatives of 12 municipalities in the neighborhood and the members of the Hydro-Electric Commission. The gathering was addressed by Sir Adam Beck, the chairman of the commission, who reviewed

the work done in the development of the Eugenia Falls system, which was begun in July, 1914, and completed in 14 months. A steel and concrete dam had been constructed 50 ft. high and half a mile in length on the Beaver River, with several smaller dams, which will impound a reserve supply of water insuring a steady production all the year round of 8,000 hp. The storage basin covers an area of 1,700 acres. This plant represents an expenditure of $596,000, and the transmission lines supplying the municipalities require a further outlay of $475,000. At present about 4,000 hp. will be supplied from the Eugenia Falls development to 12 municipalities, the principal of which are Owen Sound, Mount Forest, Orangeville, Medford, Kincardine and Grand Valley; but ultimately the output will be doubled and the number of towns and villages supplied increased to 24. When the northern district is supplied from Eugenia Falls and other plants in that section it is proposed to connect the district with the Niagara Falls system.

PERSONALS

Mr. Leonard Funk has been appointed Commissioner of Public Works of the City of Spokane, Wash.

Mr. Frank H. Ciutz, Municipal Engineer of Chambersburg, Penn., has resigned to accept a position in the city service of Philadelphia.

Mr. S. C. Odell, of Columbia, S. C., has been appointed Manager of Terminals of the Jacksonville, Fla., Municipal Docks, now approaching completion.

Mr. N. H. Bogue, formerly Assistant Engineer of the North Coast Ry., has been appointed Chief Engineer of the Oregon, California & Eastern Ry., a projected west coast railway, of which Mr. Robert E. Strahorn is President.

Mr. Thomas C. Shedd, Jun. Am. Soc. M. E., Instructor in Mechanical Engineering at Brown University, has resigned to accept a position with the Phoenix Bridge Co. at Philadelphia. He will be succeeded by Mr. Frank C. Blake.

Mr. Richard A. Hart, Assoc. M. Am. Soc. C. E., Supervising Drainage Engineer of the United States Department of Agriculture at Salt Lake City, has been appointed to take charge of the newly created branch office of the United States Office of Public Roads and Rural Engineering at Berkeley, Calif.

Prof. John C. Tracy, Professor of Structural Engineering in Sheffield Scientific School, Yale University, has been appointed to succeed the late A. J. Du Bois as head of the Department of Civil Engineering. Professor Tracy was graduated from Sheffield in 1890 and received the degree of Civil Engineer in 1892, at which time he was made an Assistant, and in 1894 an Instructor. He was appointed Assistant Professor of Structural Engineering in 1902 and Professor of Structural Engineering in 1915.

OBITUARY

Edwin Thomas Munger, General Superintendent of the Hudson & Manhattan R.R. for several years, died on Nov. 14 at his home in Paterson, N. J., at 44 years of age. He was born in Menomonie, Wis., and was graduated from the University of Wisconsin. Previous to his connection with the Hudson & Manhattan R.R. he was General Superintendent of the Metropolitan Elevated R.R. of Chicago.

Theodore Landsberg, formerly Professor of Bridge and Structural Engineering at the Darmstadt Technical College, Germany, died on Oct. 20, at the age of 68. During the time of his professorship—1880 to 1908—he made a prominent name as a leading student and investigator of structural subjects, among them influence lines and the analysis of framed domes. He received the honorary degree of doctor of engineering some years ago.

Dr. A. Veeder, a member of the American Public Health Association, and a writer on water-supply, garbage disposal and similar subjects, died at his home in Lyons, N. Y., on Nov. 16. He was born in Ashtabula, Ohio, in 1848, and was graduated from Union College in 1870. He received his medical degree from the University of Buffalo several years later. For several years Dr. Veeder was Principal of the Ives Seminary, Antwerp, N. Y. He had practiced medicine for the last 30 years in Lyons.

ENGINEERING SOCIETIES

COMING MEETINGS

THIRD ANNUAL PENNSYLVANIA WELFARE AND EFFICIENCY CONFERENCE.
Nov. 16-19. Conference in Philadelphia.

NATIONAL MUNICIPAL LEAGUE.
Nov. 17-19. Annual convention, Dayton, Ohio. Secretary, Clinton Rogers Woodruff, 705 North American Building, Philadelphia.

OHIO SOCIETY OF MECHANICAL, ELECTRICAL AND STEAM ENGINEERS.
Nov. 18-19. Annual meeting at Zanesville, Ohio. Secretary, F. E. Sanborn, Columbus, Ohio.

AMERICAN INSTITUTE OF ARCHITECTS.
Dec. 1-3. Annual convention in Washington, D. C. Executive Secretary, E. C. Kempe, The Octagon, Washington.

AMERICAN ASSOCIATION OF ENGINEERS.
Dec. 10 and 11. Convention in Chicago. Secretary, Arthur Kneisel, 29 South La Salle St., Chicago.

ASSOCIATION OF AMERICAN PORTLAND CEMENT MANUFACTURERS.
Dec. 13-15. Annual meeting in New York. Secretary, Percy H. Wilson, Bellevue Court Bldg., Philadelphia, Penn.

SECOND PAN-AMERICAN SCIENTIFIC CONGRESS.
Dec. 27-Jan. 8. Washington, D. C.

MONTANA INSTITUTE OF MUNICIPAL ENGINEERS.
Jan. 17-19. Annual meeting at Billings, Mont. Secy., C. C. Widener, Bozeman, Mont.

NINTH CHICAGO CEMENT SHOW.
Feb. 12-19. First Infantry Armory and Coliseum. Secy., R. F. Hall, 208 South La Salle St., Chicago.

SECOND NATIONAL CONFERENCE ON CONCRETE ROAD BUILDING.
Feb. 15-8. At Chicago. Secy., J. P. Beck, 208 South La Salle St., Chicago.

Engineers' Society of Western Pennsylvania announces that twice a month lists of positions open and men available will be published in its notices of meetings. The service is free and coöperation is solicited.

The Second National Exposition of Chemical Industry will be held at the Grand Central Palace, New York City, in the week of Sept. 25, 1916. The first exposition, recently held in New York City, was sufficiently successful to warrant repetition.

American Road Builders' Association—The next annual convention will be held at Pittsburgh, Penn., Feb. 22-25, 1916. In order to supply the necessary facilities, the Pittsburgh City Council recently appropriated ample funds to put Mechanical Hall of the Exposition Building in shape for this convention. It has been decided to hold the annual conventions hereafter either in January or February.

The Society of Constructors of Federal Buildings will hold its seventh annual convention at Washington, D. C., Jan. 3 to 6, 1916. It is intended to have an exhibition of building materials and appliances, an exhibition of photographs of hoisting equipment, etc. There will also be an exhibit of photographs of United States public buildings, showing the different styles of architecture that have been in vogue. Information as to the convention and the exhibits may be obtained from Irving D. Porter, 3128 Mt. Pleasant St., Washington, D. C., or from Charles Reed Marsh, secretary, Brattleboro, Vt.

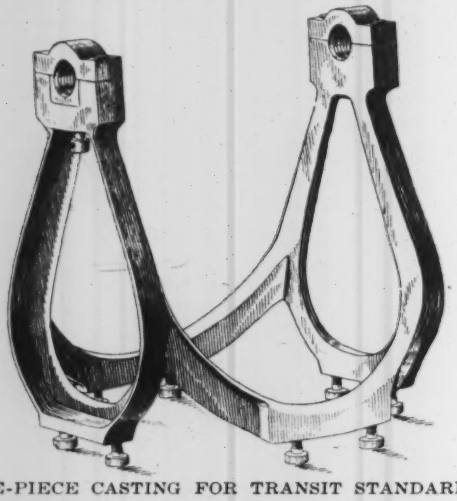
Atlantic Deeper-Waterways Association—At the eighth annual convention of the association, adjourned on Nov. 12, the following officers were elected: President, J. Hampton Moore, of Philadelphia; vice-presidents, J. H. Small, North Carolina, and W. S. Bennett, New York; F. W. Donnelly, New Jersey, and J. H. Preston, Maryland; delegates-at-large, J. Charles Linthicum, Maryland, and Frank F. Crane, Massachusetts. W. H. Schoff, Pennsylvania, was reëlected secretary-treasurer. Under a change in the constitution four vice-presidents-at-large and two delegates-at-large are elected.

The American Civic Association will hold its eleventh annual convention at the New Willard Hotel, Washington, D. C., Dec. 28 to 31. The convention will be held in conjunction with the Second Pan-American Scientific Congress. The program will include addresses and discussions relating to city, town and country planning; to residential and garden-suburb development; to national and state parks; to Niagara Falls preservation; to the abatement of the billboard nuisance. The Scientific Congress will treat the scientific aspects of several of the topics just mentioned. The secretary of the association is Richard B. Watrous, Union Trust Building, Washington D. C.

Appliances and Materials

One-Piece Trussed Transit Standards

A new line of precise transits is being introduced by W. & L. E. Gurley, Troy, N. Y. The principal features of the new instruments are the standards and the lower plate castings. The standards are cast in one piece, as shown in the accompanying sketch; the casting is of bronze and of angle cross-section. It is fastened to the plate casting by eight



ONE-PIECE CASTING FOR TRANSIT STANDARDS

fillister-headed screws instead of four screws, as in the usual type of transit standard. The plate castings have 24 to 26 radial ribs to give rigidity. The new instrument, with a 6.7-in. horizontal circle, weighs 18½ lb., or 29½ lb. with the tripod. The list price is $220. The same construction is also used for an instrument with a 6.25-in. horizontal circle, which is made in two weights—16½ and 27½ lb. The list price is the same.

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Gang Drilling Machine for Structural Steel

There is in the Machinery Building at the Panama-Pacific Exposition a new gang drill which is claimed to bore a larger number of holes than any such drill hitherto made. A front view of the machine is shown herewith. The drills are rotated by two sprocket chains, which engage with sprocket wheels mounted on the spindles of each alternate drill head. This enables the drill spindles to be brought very close together without requiring the driving sprocket to be of very small diameter. Each drill head may be moved independently of the other and is clamped to the horizontal slide by a hand lever. The frame which carries the drill heads is stationary and the table on which the plate or other material to be drilled is placed is raised by automatic mechanism to give any desired feed. The drills can be spaced as accurately as in any gang drilling machine, and provision is made for automatic feed of cooling and lubricating liquid. In the present machine the closest spacing possible is about 3 in., but with slight alterations in design it will be possible to reduce this. The chuck takes standard sizes of twist drills and is arranged to permit their quick removal from the machine.

HILL GANG DRILL FOR STRUCTURAL WORK

The inventor of the machine is Aaron Hill, of Los Angeles, and the first machine was built at shops in that city. The machine is being placed upon the market by the Hunt Engineering and Sales Agency, Baker-Detwiler Building, Los Angeles, Calif.

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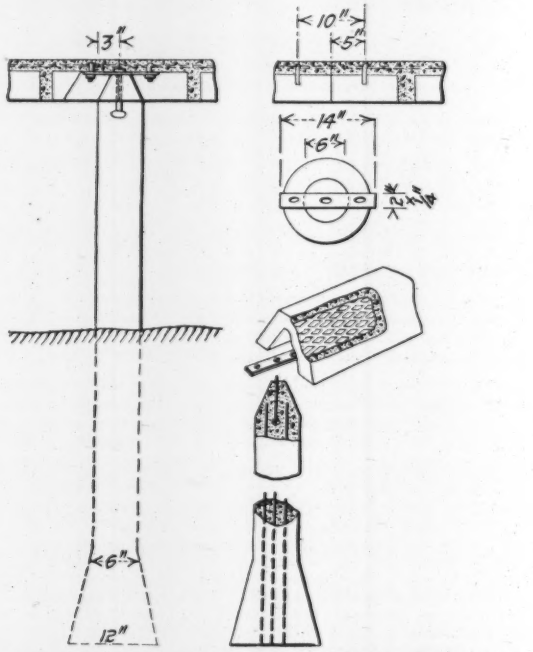
Patented Concrete Guard Rail

A reinforced-concrete guard rail for use on highways has been patented by H. E. Dillon, Construction Engineer, Olean, N. Y., who manufactures the steel-unit reinforcement and the special forms for molding the rail and posts. The rail itself is of inverted V-shape cross-section and is reinforced with diamond-mesh wire cloth. It is made in 8-ft. lengths and is connected to the posts by slotted iron strips attached to the under side of the rail. Through these strips bolts in the top of the posts are passed and held by nuts. While the forms are in sections, the rail may be cast monolithic if desired. The posts are round, 6 in. in diameter, spread at the



DILLON REINFORCED-CONCRETE GUARD RAIL

base to 12 in. The tops of the posts are tapered to avoid frost damage to the rail and to give it a firm bearing under all conditions. The reinforcement forms a framework which would stand of itself, thus providing the completed structure with considerable strength against shock. The inventor claims that this rail may be erected in place at approximately the cost of a first-class wooden guard rail.

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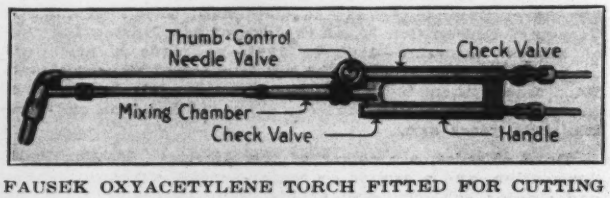
Oxyacetylene Welding and Cutting Outfits

A newly improved line of cutting and welding outfits, designed by A. J. Fausek, is being made by the Modern Engineering Co., St. Charles and 14th St., St. Louis, Mo. They are variously arranged for concerns all the way from small smiths and manufacturers up to the largest railways, welders and wreckers. The important item, besides hose, gages, etc., is the torch—which is shown in the view as fitted for cut-



FAUSEK OXYACETYLENE TORCH FITTED FOR CUTTING

ting. Patent check-valves prevent flow from one gas line to the other. The handle is a one-piece light-alloy casting. To attach the cutter, the welding arm and tip are removed from the mixing chamber, and the cutting head, with its oxygen-control valve and conduit, is put on. The tank-hose connections are not disturbed.